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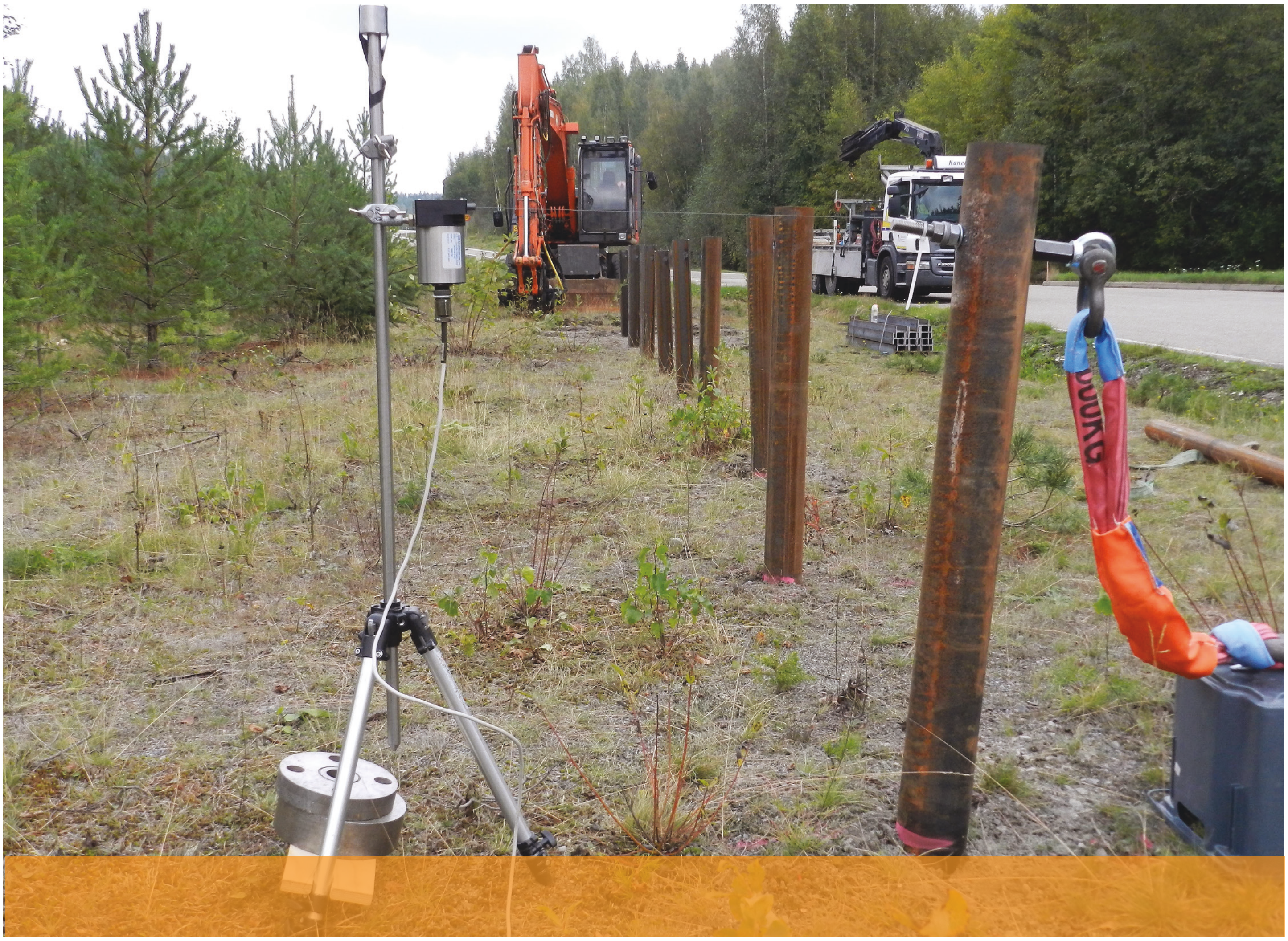
An In-situ Investigation



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Abstract

As the European standard EN 1317 (1-5) mandates certain full-scale tests on the elements that are used in safety barriers, the present study focuses on the testing of lateral load capacity of safety barrier piles (section 5.1 of prEN 1317-5 E).

This report summarises the findings from a series of in-situ lateral load capacity test on piles that are intended to be used in road barrier applications. Tests were conducted in tubular (O-piles) and HEB (H-piles) pile profiles in three different soil conditions. The embedment depths of the piles varied between 1 m and 3 m. In addition, piles were also tested under wet and recompacted conditions in selected sites.

The objectives of this study are:

i. To determine the average resistance values (F20, F200 & F_{max}) of O- and H-pile profiles
ii. To investigate the resistance values achieved with H-pile against the O-pile in different soils
iii. To evaluate the resistance values at different pile embedment depths.

According to the average full-scale lateral pull test results, piles with a embedment length of 2 metre gave the best bending resistance. In case of O-profile piles, the wall thickness should be 8 mm in order to avoid material failure. In general, the H-profile yielded a better bending resistance in moraine and slightly better in fragmented rock; however, in crushed rock there was no significant difference between the bending resistances of H and O-profiles. It was concluded that the poor compaction caused major discrepancies compared to the impact of degree of saturation on the test results. It is worthwhile to mention that the number of pile profiles tested in this study was relatively small and hence it is difficult to draw a significant conclusion; however, the data presented in this report may serve as a benchmark for further investigations.

Gowthaman Sinnathamby ja Leena Korkiala-Tanttu: Kaidepylväiden vaakasuuntainen kapasiteetti maastomittausten perusteella. Liikennevirasto, tekniikka- ja ympäristöosasto. Helsinki 2018. Liikenneviraston tutkimuksia ja selvityksiä 44/2018. 42 sivua ja 3 liitettä. ISSN-L 1798-6656, ISSN 1798-6664, ISBN 978-952-317-599-0.

Avainsanat: pylväsperustus, vaakasuuntainen kapasiteetti, vetokoe,

Tiivistelmä

Tämä tutkimus esittää kaidepylväiden täydenmittakaavan vetokokeiden tuloksia ja analysoi niitä. Kokeet on tehty standardin EN 1317 (1-5) mukaisesti ja niiden avulla on arvioitu eri kaidepylväiden vetosuuntaista vetokapasiteettia. Kokeita tehtiin pyöreille O-profiileille ja HEB (h-profiilileille) kolmassa erilaisessa maaryhmässä. Pylväiden upotussyvyydet vaihtelivat välillä 1–3 metriä. Sen lisäksi yhdessä kohteessa testattiin kosteiden ja uudelleen tiivistettyjen täyttöjen eroa kuiviin olosuhteisiin.

Tutkimuksen tavoitteena oli:

iv. Määrittää keskimääräiset vetokapasiteetit 20 mm, 200 mm ja maksimisiirtymille sekä O- että H-profiileille (F_{20}, F_{200} & F_{max}).
v. Vertailla O- ja H-profiilien vaikutusta taivutusvastukseen eri maaryhmissä.
vi. Arvioida upotussyvyyden vaikutusta taivutusvastukseen.

Keskiarvostettujen koetulosten mukaan parhaaksi upotussyvyydeksi saatiin 2 metriä. Taivutusvastus ei enää juurikaan kasvanut paalun upotussyvyyden kasvaessa. Paalumateriaalin parhaaksi seinämävahvuudeksi todettiin 8 mm, sitä ohuemmilla seinämillä tapahtui paalun rakenteellinen murtuminen. H-profiili osoittautui selvästi paremmaksi kuin O-profiili moreenissa, hieman paremmaksi louhessa, mutta murskeessa profiilin muodolla ei näyttänyt olevan vaikutusta. Kosteiden olosuhteiden vaikutusta testattiin yhdessä kohteessa, mutta tulosten mukaan näyttäisi siltä, että huonommalla tiivistyksellä olisi ollut suurempi vaikutus kuin kosteuden kasvulla. Koemäärät olivat melko vähäisiä, joten koetuloksia on pidettävä suuntaa antavina.

Gowthaman Sinnathamby och Leena Korkiala-Tanttu: Räckenstolpars horisontella kapacitet baserat på terrängmätning. Trafikverket, teknik och miljö. Helsingfors 2018. Trafikverkets undersökningar och utredningar 44/2018. 42 sidor och 3 bilagor. ISSN-L 1798-6656, ISSN 1798-6664, ISBN 978-952-317-599-0.

Sammanfattning

Denna studie presenterar och analyserar resultaten för fullskaliga dragprov på räckenstolpar. Testen har gjorts enligt standarden EN 1317 (1–5), och med hjälp av dem har dragriktningens dragkapacitet för olika räckenstolpar utvärderats. Runda O-profiler och HEB (h-profiler) testades i tre olika terränggrupper. Stolparnas nedsänkningsdjup varierade mellan 1 och 3 meter. På ett objekt testades dessutom skillnaden mellan fuktig och återtätad utfyllnad i torra förhållanden.

Målet med undersökningen var att:

i. Fastställa den genomsnittliga dragkapaciteten för 20 mm, 200 mm och den maximala förskjutningen samt för O- och H-profiler (F20, F200 & F_{max}).
ii. Jämföra O- och H-profilers påverkan på böjmotståndet i olika terränggrupper.
iii. Bedöma nedsänkningsdjupets effekt på böjmotståndet.

Enligt de genomsnittliga testresultaten var det bästa nedsänkningsdjupet 2 meter. Böjmotståndet ökade inte mycket när stolpens nedsänkningsdjup ökade. Stolpmaterialets bästa väggtjocklek var 8 mm, de tunnare väggarna fick strukturella sprickor i stolparna. H-profilen visade sig vara klart bättre än O-profilen i morän, något bättre i sprängsten, men i kross verkade profilens form inte ha någon betydelse. Betydelsen av fuktiga förhållanden testades på ett objekt, men enligt resultaten verkar det som att sämre tätning skulle ha större betydelse än fuktökning. Testvolymerna var ganska låga, så testresultaten bör ses som vägledande.

Foreword

The project group consisted of experts from the Finnish Transport Agency (Liikennevirasto) and researchers from Aalto University.

From the Finnish Transport Agency the following experts were involved: Kari Lehtonen and Veli-Matti Uotinen. From Aalto University, the following researchers were involved: Gowthaman Sinnathamby, Leena-Korkiala Tanttu and the Laboratory Chief Engineer Veli-Antti Hakala. Jukka Piironen and Petri Peltonen from Aalto University provided technical assistance in setting up the in-situ measurement system. The pile driving was conducted by *Kanerva Oy Kaide ja Kuljetus*.

Helsinki, August 2018

Finnish Transport Agency
Engineering and Environment

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Abbreviations

FR	Fragmented Rock
Moraine	Moraine Type Soil
CA	Crushed Aggregate
O-pile	Tubular pile
H-pile	HEB pile
BTA	Barrier Testing Area
STA	Support Testing Area

D	[m]	pile diameter
EI	[kNm²]	bending stiffness of the pile section
F		factor of safety = 1.35, for displacement on the ground <30 mm (based on calculations)
F20	[kN]	Resistance corresponding to 20 mm displacement at the top of the pile
F200	[kN]	Resistance corresponding to 200 mm displacement at the top of the pile
F_{max}	[kN]	Ultimate / Maximum Resistance
H	[kN]	lateral load
SF_{max}	[mm]	Maximum displacement
$K_p\gamma$	[kN/m³]	soil parameter (Table 4)
K_p		Passive earth pressure coefficient
L	[m]	embedment depth
W	[m³]	section modulus
γ	[kN/m³]	unit weight of the soil
e	[m]	eccentricity of the lateral load from the ground level
z	[m]	depth of embedment
σ_u	[kN/m²]	maximum permissible stress of the pile in the ultimate limit state
k1		slope form factor (Luukkonen, 2015)

1 Introduction

Installation of road restraint systems on critical sections of roads ensures enhanced road safety. In essence, road restraint systems are put in place to redirect and/or contain errant vehicles safely or to reduce the severity of vehicle impact with a more resistive object, for the benefit of the occupants and other road users, including pedestrians (prEN 1317-5 E). Vehicle restraint systems shall include safety barriers, crush cushions, terminals, transitions, safety barriers combined with motorcyclists protection and vehicle parapets combined with pedestrian parapets (prEN 1317-5 E). Metal piles with tubular and HEB profiles are two of the most commonly used profiles in safety barriers across Europe.

As the European standards EN 1317 (1-5) mandate certain full-scale tests on the elements used in safety barriers, the present study focuses on the testing of safety barriers (section 5.1 of prEN 1317-5 E), particularly on the testing of ground (section 5.1.6 of prEN 1317-5 E).

This report summarises the findings from a series of lateral load capacity tests on piles that are intended to be used in road barrier applications. The tests were carried out on tubular (O-profile) and HEB (H-profile) piles that were driven into various backfills at different embedment depths. Additionally, the piles were also tested under wet weather conditions.

The following are the objectives of this study:

i. To determine the average resistance values (F20, F200 & F_{max}) of O- and H-pile profiles
ii. To investigate the resistance values achieved with H-pile against the O-pile in different soils
iii. To evaluate the resistance values at different pile embedment depths.

2 Testing Programme

2.1 Test locations and ground conditions

The pile lateral load capacity tests were conducted in three different sites in Finland, namely: Vantaa (GPS: 60.296248, 24.957039), Mäntsälä (GPS: 60.693962, 25.398953) and in Pori (GPS: 61.453007, 21.801054) (Fig. 1). A number of tubular and HEB piles were tested in all sites with various backfills (Tables 1–3b).

Figure 1. Test sites: (a) Vantaa, (b) Mäntsälä, and (c) Pori (Source: Google maps).

2.1.1 Vantaa

In Vantaa, a total of 21 piles were installed in a fragmented rock embankment and tested on 02.10.2014 (Table 1). The fragmented rocks in this site were relatively large with fragments as big as 250 mm. Fragmented rock was overlain by a 400 mm thick crushed aggregate layer. The site layout and pile installation plan in Fragmented rocks (Vantaa) are shown in Figure 2.

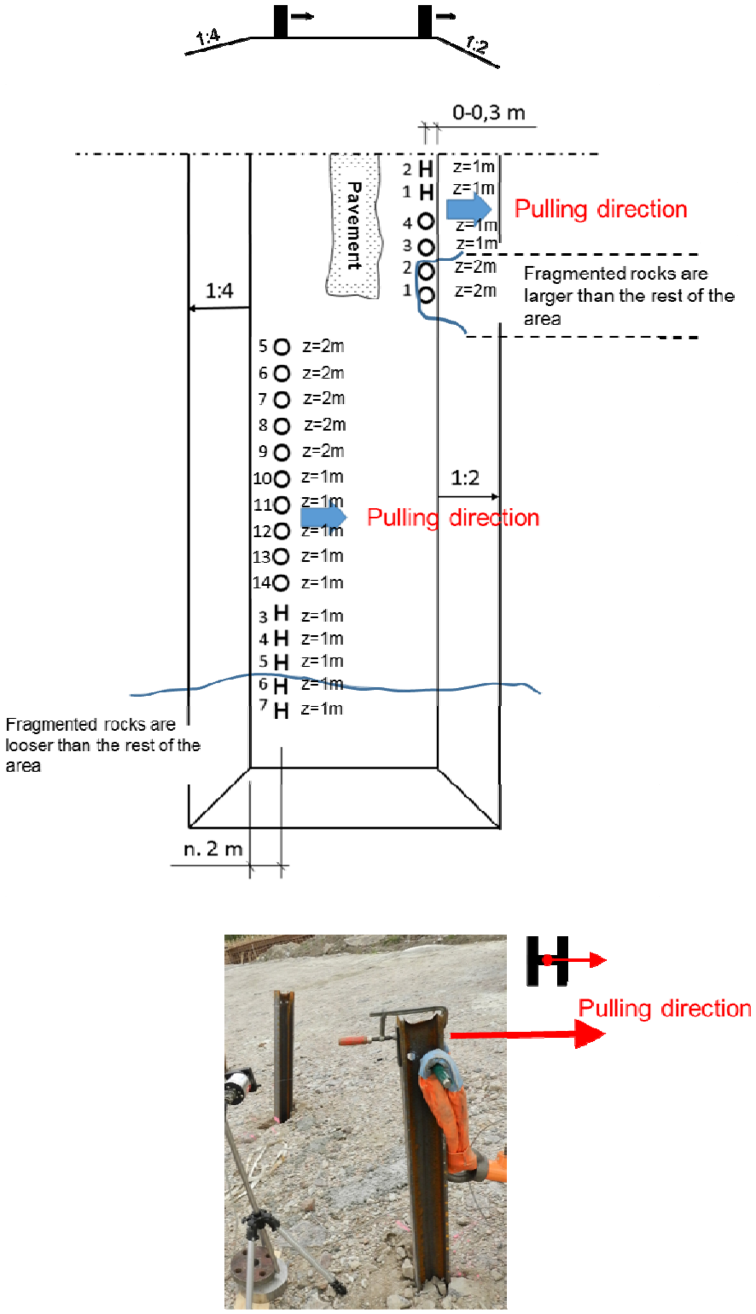


Figure 2. *Pile installation layout in Fragmented Rock in Vantaa.*

Piles were installed on either side of the embankment. The embankment had different sloping angles as indicated in Figure 2. For practical reasons, the slope with 1:4 gradient over a 2 m length is assumed to be flat. It is worthwhile to mention that this crushed aggregate layer was no uniform and some areas did not have any crushed aggregate cover at all. In these locations, the fragmented rock was exposed to the ground surface and the piles were driven directly into it. The non-uniform ground conditions with exposed fragmented rocks and loose fragments are indicated in Figures 2 & 3a. Due to transducer malfunction, the force-displacement data for pile O-1 was not recorded.

All the H-piles had an embedment depth of 1m while the O-profile had embedment depths of 1m and 2m. Piles were categorised based on the direction of applied load, namely; pulling towards a flat ground and pulling towards a 1:2 slope.

Table 1. Summary of installed piles in Fragmented Rock in Vantaa.

ID	O-1	O-2	O-3	O-4	O-5	O-6	O-7	O-8	O-9	O-10	O-11	O-12	O-13	O-14	H-1	H-2	H-3	H-4	H-5	H-6	H-7
Length (m)	2	2	3	3	3	3	3	3	3	2	2	2	2	2	2	2	2	2	2	2	2
Depth of installation (m)	1	1	2	2	2	2	2	2	2	1	1	1	1	1	1	1	1	1	1	1	1
Type	O	O	O	O	O	O	O	O	O	O	O	O	O	O	H	H	H	H	H	H	H

Note: O – Tubular pile; H – HEB pile

Figure 3. Ground conditions at pile installation sites: a) Fragmented Rock (FR) in Vantaa with non-uniform crushed aggregate cover, b) relatively uniform Moraine Type Soil in Mäntsälä, and c) Crushed Aggregate (CA) in Pori Barrier Testing Area (BTA).

2.1.2 Mäntsälä

Mäntsälä site was composed of a Moraine type soil on a flat ground (Fig. 3b). A total of 15 piles (10 tubular + 7 HEB) were installed and tested on 10.09.2014. The soil in Mäntsälä is categorised as Moraine type soil (srHkMr/HkMr/hkSrMr/siHkMr) according to the European standard (EN ISO 14688-2) and the old Finnish guidelines (Korhonen et al. 1974) (Fig. 5).

Table 2. Summary of installed Piles in Moraine Type Soil in Mäntsälä.

ID	O-1	O-2	O-3	O-4	O-5	O-6	O-7	O-8	O-9	O-10	H-1	H-2	H-3	H-4	H-5
Length (m)	3	3	3	3	3	2	2	2	2	2	2	2	2	2	2
Depth of installation (m)	2	2	2	2	2	1	1	1	1	1	1	1	1	1	1
Type	O	O	O	O	O	O	O	O	O	O	H	H	H	H	H

2.1.3 Pori

The ground in Pori was composed mainly of crushed aggregates (CA) (Fig. 3c) which is categorised as gravel (Gr). Tests were conducted on 22-23.09.2015. A total of 32 piles were tested in Pori. The testing area in Pori consisted of two designated sections, namely; a Barrier Testing Area (BTA) and a Support Testing Area (STA) (Fig. 4). The test area was constructed about fifteen years ago. During the construction, the ground was compacted with heavy compactor plates weighing around 200 kg and the compaction was done in 0.3m thick layers. Upon the completion of the compaction, the BTA was covered with hot mixed asphalt and compacted. In 2015, a small section was exposed (shown as BTA and STA in Fig. 4) by removing the top asphalt cover. The STA area was also built at the same time as BTA and underwent a similar treatment as in BTA, though the ground was compacted with a lighter vibratory plate (about 80 kg) and in 0.3m thick layers.

A series of O and H-piles with varying embedment depths were tested at the BTA. At the STA, only H-piles were tested with original and new crushed aggregate backfills. Grading curves of the original and new crushed aggregate are shown in Figure 5. The crushed aggregate in STA and BTA are similar and also identical to the new crushed aggregate that was used in STA.

In addition to the natural dry ground, some H-piles (H2.2m-1M, H2.2m-2M, H2.2m-3M, H2.2m-4M and H2.2m-5M) were tested under wet ground / backfill conditions at the BTA by wetting the soil prior to testing. A series of tests on H-piles (PT-H2.2m-1 to PT-H2.2m-6) were conducted at the support testing area (STA). Three different backfill cases were considered in these tests.

i. At first, two H-piles (PT-H2.2m-1 & PT-H2.2m-1) were driven into the original crushed aggregates (Curve S3) and subsequently tested. This case was originally compacted as in case iii below, but rain and time have compacted it further.

ii. In the second case, the original crushed aggregate (Curve S3) was removed, and then put back and compacted by vibratory plate and dampened. Two H-piles (PT-H2.2m-3 and PT-H2.2m-4) were driven into the backfill and tested subsequently.

iii. In the third case, the original backfill was removed and replaced by new crushed aggregate (Curve S4) with its natural moisture content. Two piles (PT-H2.2m-5 and PT-H2.2m-6) were then installed and tested.

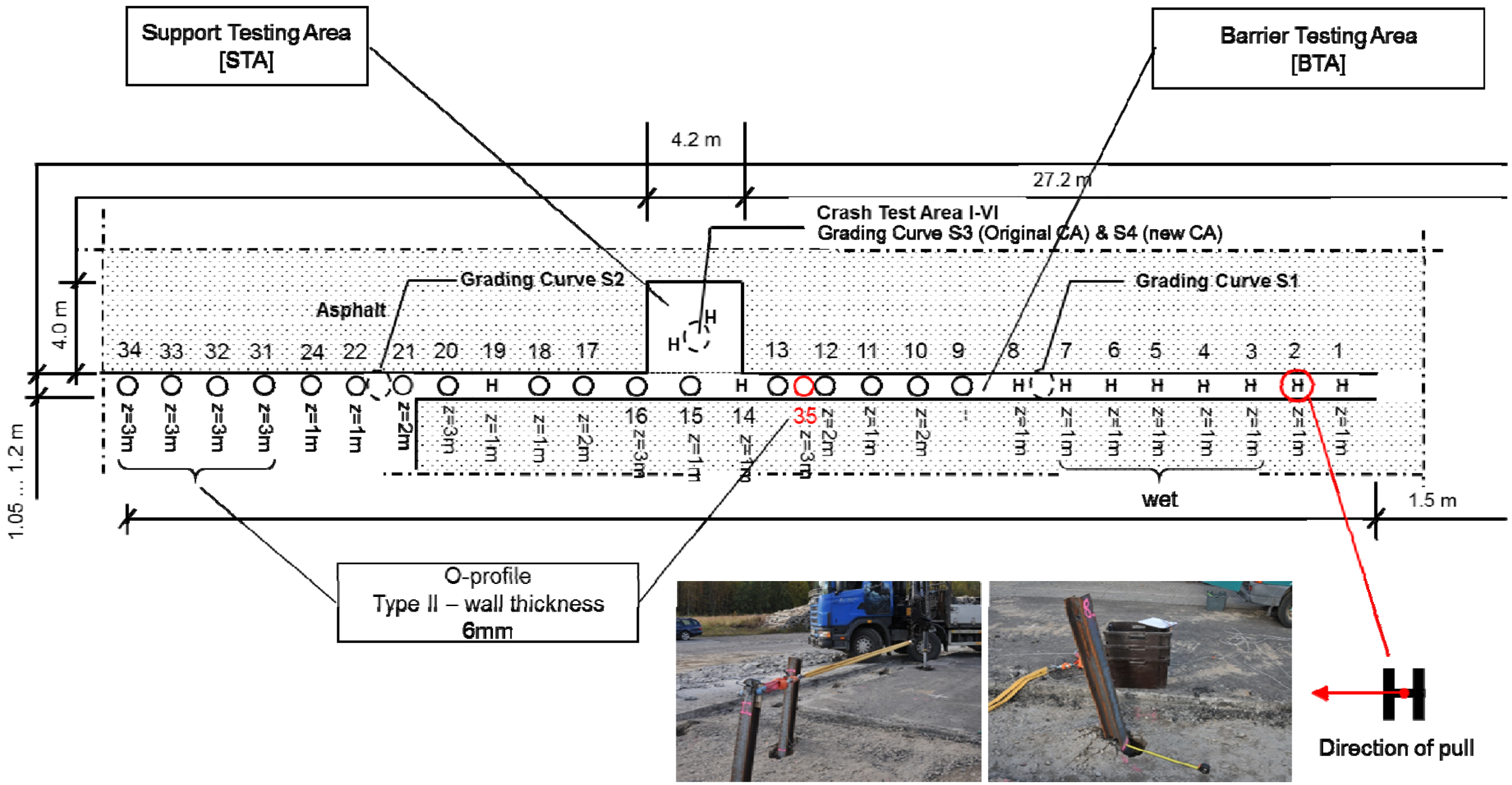


Figure 4. *Pile installation layout in Crushed Aggregate (CA) in Pori: Test area consisted of two designated areas, namely: a Barrier Testing Area (BTA) and a Support Testing Area (STA).*

Table 3a. Summary of O-piles installed in Crushed Aggregate in Pori.

No. in layout	16	20	10	12	17	21	11	15	18	22	24	31	32	33	34	35
Beam ID	O4m-3	O4m-4	O3m-1	O3m-2	O3m-3	O3m-4	O2m-1	O2m-2	O2m-3	O2m-4	O2m-5	O4m-I	O4m-II	O4m-III	O4m-IV	O4m-V
Length (m)	4	4	3	3	3	3	2.2	2.2	2.2	2.2	2.2	4	4	4	4	4
Depth of installation (m)	3	3	2	2	2	2	1	1	1	1	1	3	3	3	3	3

Table 3b. Summary of H-piles installed in Crushed Aggregate in Pori.

No. in layout	1	2	8	14	19	3	4	5	6	7	I	II	III	IV	V	VI
Beam ID	H2.2m-1	H2.2m-2	H2.2m-3	H2.2m-4	H2.2m-5	H2.2m-1M	H2.2m-2M	H2.2m-3M	H2.2m-4M	H2.2m-5M	PT-H2.2m-1	PT-H2.2m-2	PT-H2.2m-3	PT-H2.2m-4	PT-H2.2m-5	PT-H2.2m-6
Length (m)	2.2	2.2	2.2	2.2	2.2	2.2	2.2	2.2	2.2	2.2	2.2	2.2	2.2	2.2	2.2	2.2
Depth of installation (m)	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1

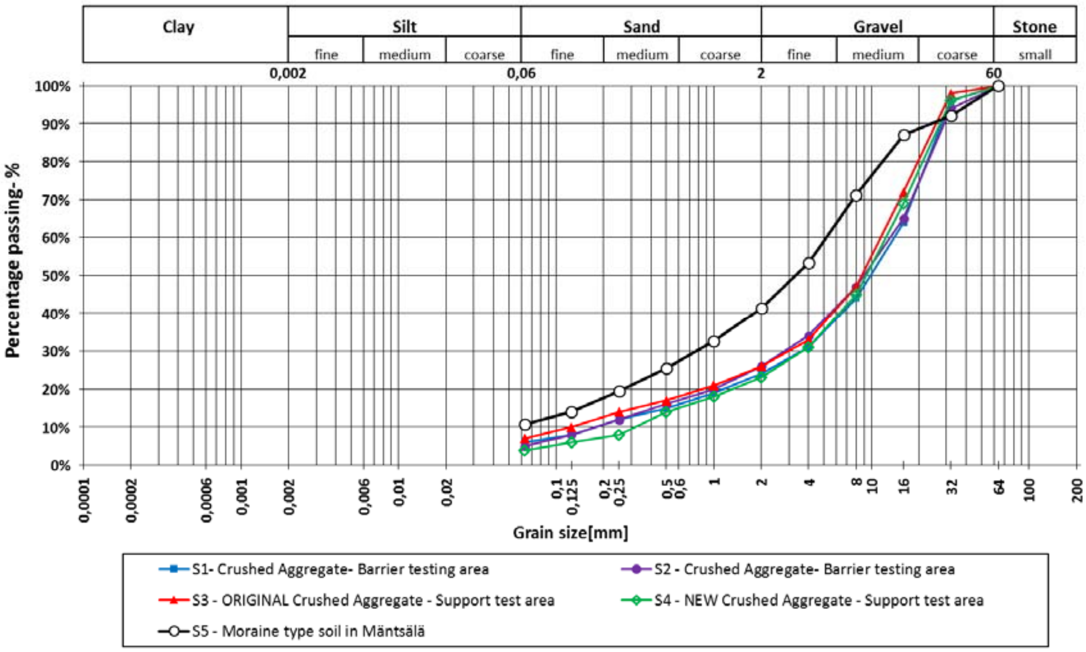


Figure 5. Particle size distribution of the soil materials: Moraine Type Soil from Mäntsälä (Curves S1-S4) and Crushed Aggregate from Pori (Curve S5) (see Appendices 2 & 3 for further sieving curves).

2.2 Cases

Based on the pile profiles, embedment depth and soil conditions, the following cases are identified from the lateral load capacity test series. Results from each site are analysed based on these cases. Unless otherwise mentioned, the wall thickness of the O-profile piles is 8 mm.

In Pori

Case P1: O – Profile: Depth 1 m (z = 1m): in Crushed Aggregate (CA) Curve S1 (S2/S3) in barrier testing area

Case P2: O – Profile: Depth 2 m (z = 2m): in Crushed Aggregate (CA) Curve S2 (S1/S3) in barrier testing area

Case P3: O – Profile: Depth 3 m (z = 3m): in Crushed Aggregate (CA) Curve S2 (S1/S3) in barrier testing area

Case P3*: O – Profile: Depth 3 m (z = 3m): in Crushed Aggregate (CA) Curve S2 (S1/S3) in barrier testing area (nominal wall thickness 6.3 mm)

Case P4a: H – Profile: Depth 1 m (z = 1m): in Crushed Aggregate (CA) Curve S1 (S2/S3) in barrier testing area

Case P4b: H – Profile: Depth 1 m (z = 1m): in Wet Crushed Aggregate (CA) Curve S1 (S2/S3) in barrier testing area

Case P5a: H – Profile: Depth 1 m (z = 1m): in Crushed Aggregate (CA) Curve S3 (CR) in support testing area

Case P5b: H – Profile: Depth 1 m (z = 1m): in Crushed Aggregate (CA) Curve S3 wet (CR) in support testing area

Case P5c: H – Profile: Depth 1 m (z = 1m): in Crushed rock (CR) Curve S4 (new CR) in support testing area

Note: Curves S1 & S2: CA from BTA; Curve S3 – Original CA from STA; Curve S4 – New CA from STA

Mäntsälä

Case M1: O – Profile: Depth 1 m (z = 1 m) in natural moraine type soil Curve S5

Case M2: O – Profile: Depth 2 m (z = 2 m) in natural moraine type soil Curve S5

Case M3: H – Profile: Depth 1 m (z = 1 m) in natural moraine type soil Curve S5

Vantaa

Case V1a: O – Profile: Depth 1 m (z = 1 m) in larger than usual fragmented rock; pulled towards 1:2 slope

Case V1b: O – Profile: Depth 1 m (z = 1 m) in fragmented rock; pulled towards flat ground

Case V2a: O – Profile: Depth 2 m (z = 2 m) in fragmented rock; pulled towards 1:2 slope

Case V2b: O – Profile: Depth 2 m (z = 2 m) in fragmented rock; pulled towards flat ground

Case V3a: H – Profile: Depth 1 m (z = 1 m) in fragmented rock; pulled towards 1:2 slope

Case V3b: H – Profile: Depth 1 m (z = 1 m) in fragmented rock; pulled towards flat ground

Case V3c: H – Profile: Depth 1 m (z = 1 m) in loose fragmented rock; pulled towards flat ground

In order to meet the objectives of this study, the following tasks were carried out on all cases listed above:

Task 1: Calculate the average resistance values (F20, F200 & Fmax) for each of the cases mentioned above

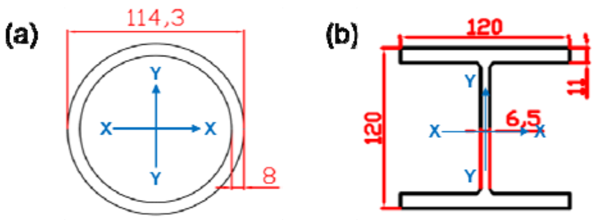
Task 2: Compare the resistance values achieved with H-Profile against the O-Profile in different backfills / soils at similar depths

Task 3: Compare the resistance values with depth within the same backfill / soil

2.3 Installation and Testing

2.3.1 Piles

The sections of the tested pile profiles and some of the key properties of the steel are listed below in Figure 6. For steel grade S420MH, only nominal yield strength values are available. For other steel grades, the yield stress values supplied by the manufacturer are applied. The HEB profiles were pulled along the x-axis. The length of the piles varied between 1 m and 4m, however, in all cases around 1 m were left above the ground after installation. Thus, the embedment depth was varied (Tables 1-3b).



Profile	O		HEB-120
	Type I – 8mm wall	Type II – 6.3 mm wall*	
Steel Grade	NLERWS460MH S460MH	S420MH	S355J2+AR
Length (m)	2 / 3 / 4	4	2 / 2.2
Depth of embedment, z (m)	1 / 2 / 3	3	1 / 1.2
Moment of inertia, I_{xx} (cm^4)	379.492	312.714	837.789
Moment of inertia, I_{yy} (cm^4)	379.492	312.714	317.024
Young's Modulus, E (GPa)	210	210	210
Yield Strength, ReH (N/mm^2)	495	420 (nominal value)	383
Tensile Strength, Rm (N/mm^2)	600	600	563
Section modulus, W_z (cm^3)	60.83	52.95	144.1

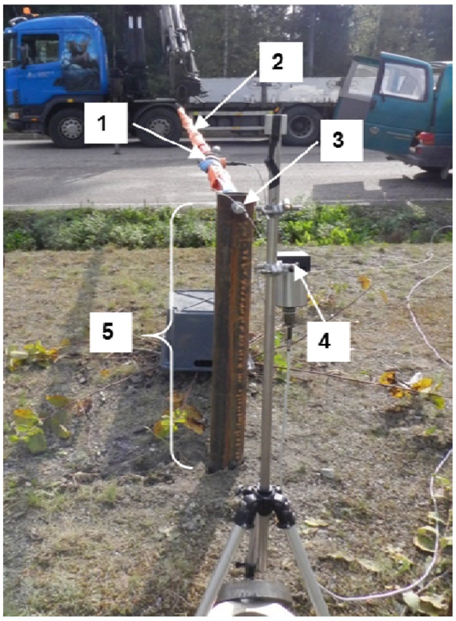
Figure 6. Tested pile profiles: (a) Tubular section (O-profile), and (b) HEB section (H-profile).

2.3.2 Installation and Testing

The piles were installed vertically in the backfill with a hydraulic hammer. The lateral pulling force was then applied on the piles through a steel cable that was attached to the pile by a high strength bolt. Force and displacement transducers were fitted as required (Fig. 7). Guidelines given in standard prEN 1317-5:2013 (E) was followed during the tests. A force increment of 0.001 kN at a speed of 10 m/s was used in all the tests. Tests were ceased when one of the following conditions was reached:

a) The pile deflected more than 0.4 m at loading/measuring position
b) A force higher than 45 kN was achieved at loading/measuring position.

Force- displacement data was recorded automatically for further analysis. The minimum acquisition load spacing was 1 kN and the deflection spacing was ± 10 mm. Tolerances were 0.2 kN for the force and 1 mm for the displacement. The final shape of the pile and height of the measuring point was also recorded.



1. **Force transducer**
2. Pulling steel cable
3. High strength bolt
4. Displacement transducer
5. h - Height of the beam above ground level **(h ≈ 1)**

Figure 7. Pile lateral capacity test setup.

3 Results

3.1 Theoretical Background

3.1.1 General

According to Broms (1964), failure of a laterally loaded pile takes place either when the maximum bending moment in the loaded pile reaches the ultimate or yield resistance of the pile section or when the lateral earth pressure reach the ultimate lateral resistance of the soil along the total length of the pile (Broms, 1964).

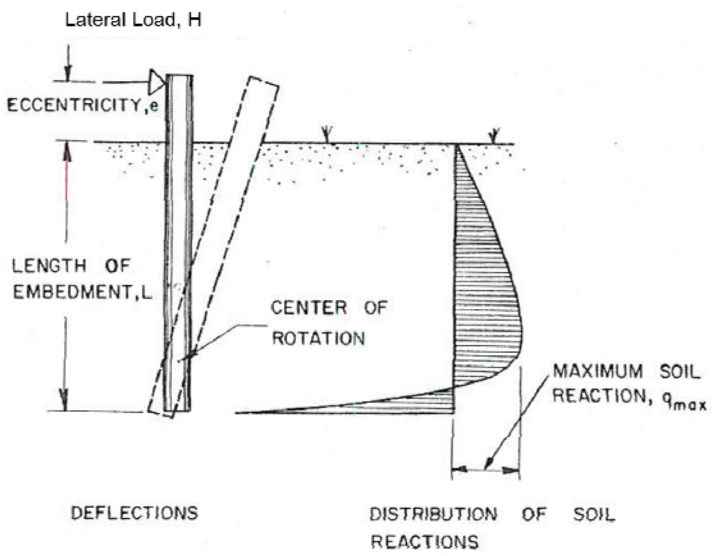


Figure 8. Failure mode of a short free-headed pile (Broms, 1964).

Figure 8 illustrates the failure mode of a typical short unrestrained pile in cohesionless soil. In this case, failure occurs when pile rotates as a single unit with respect to a point located close to its toe. In other words, failure happens due to lateral earth pressure reaches the ultimate lateral resistance of the soil and the ground fails (Broms 1964).

In case of a long unrestrained pile, failure takes place when the maximum bending moment exceeds the yield resistance of the pile section and a plastic hinge forms at the section of maximum bending moment (Fig. 9). In other words, failure occurs due to the failure of the pile (Broms 1964).

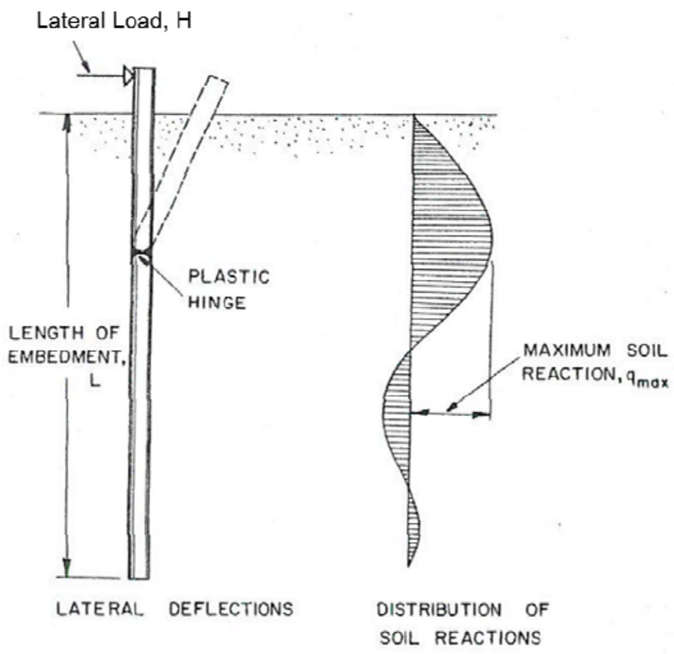


Figure 9. Failure mode of a long free-headed pile (Broms, 1964).

According to Finnish Road Administration 2001 the design of the above piles can be done in accordance with the design formulae of rigid foundations (in Section 3.1.2), if

$$\frac{(EI)^{0,6} \cdot (K_P\gamma)^{0,3}}{18 \, kN^{-0,1} \, m^{-0,7}} \geq H \cdot e \tag{1}$$

where:

EI - stiffness of the pile section [kNm²]
$K_p\gamma$ - soil parameter [kN/m³] (Table 4)
K_p - Passive earth pressure coefficient
γ - unit weight of the soil [kN/m³]
H - lateral load [kN]
e - eccentricity of the lateral load from the ground level [m]

The soil parameters used in this report are extracted from "Sivukuormitetut pilari-perustukset - Suunnitteluvaiheen ohjaus: *Taulukko 1*" (Finnish Road Administration, 2001) and can be found in Table 4.

3.1.2 Designing a rough pile and columnar foundation / Calculation of lateral load capacity of short and long piles

Aforementioned, when designing a rigid pile in frictional soil, the following failure modes should be considered:

a) Due to the failure of the surrounding soil / upper permissible displacements
b) Due to the failure of the pile itself

The design principles of rigid piles used in this report are based on the theory proposed by Bengt Broms (Broms, 1964); however, the modified formulas are adopted in calculations so that the calculated displacements on the ground surface do not exceed 30 mm (Finnish Transport Agency, 2001).

Finnish Road Administration "Sivukuormitetut pilariperustukset- Suunnitteluvaiheen ohjaus" (Finnish Road Administration, 2001) sets out the following guidelines for determining the lateral load capacity, H_{SHALL} of a rigid column foundation.

The lateral load capacity (maximum permissible horizontal load) of a pile is obtained by selecting the lowest of the results of A and B (Finnish Road Administration 2001).

Where the following parameters are know:

D - pile diameter
L - embedment depth
W - bending resistance
σu - maximum permissible stress of the pile in the ultimate limit state
e - eccentricity of the lateral load above the ground level
K_P - passive pressure coefficient
γ - unit weight of the soil
k1 - slope form factor (Luukkonen, 2015)
F- factor of safety = 1.35, for displacement on the ground <30 mm (based on calculations)

A) The maximum lateral load where the ground does not fail

The maximum permissible lateral load, H_{SALL} when ground failure is considered as 30 mm at the ground level.

$$H_{SALL} = \frac{0{,}375 \cdot k_1 \cdot K_P \cdot \gamma \cdot D \cdot L^3}{1{,}2 \cdot e \cdot F} \qquad \textbf{(2)}$$

B) The maximum lateral load for which the pile does not fail

The maximum permissible lateral load, H_{SALL} for maximum bending moment on the foundation.

$$H_{SALL} = \frac{\sigma_u \cdot W}{e + 0{,}55\sqrt{\frac{\sigma_u \cdot W}{(e + 0{,}5\mathrm{m})(D \cdot K_P \cdot \gamma)}}} \qquad \textbf{(3)}$$

C) The maximum lateral load for which the foundation is so rigid that equations A and B are valid

In this case, the maximum permissible lateral load, H_{SALL} can be calculated from equation 4 in accordance with the criteria in equation 1.

$$H_{SALL} = \frac{(EI)^{0,6} \cdot (K_p \gamma)^{0,3}}{e \cdot 18\, kN^{-0,1} m^{-0,7}} \quad \textbf{(4)}$$

Table 4. Summary of soil properties used in the calculations (Finnish Transport Agency, 2001).

Soil type code and description (no special surrounding filling)		Friction angle ϕ, °	Unit weight, γ, kN/m³	Coefficient of passive earth pressure, K_p	Module of sub-grade reaction, n_h, MN/m³	γK_p, kN/m³
P1	Extremely well compacted crushed or fragmented rock, driven or bored pile	46	23,2	6,13	48	142
P2	Dense crushed or fragmented rock	40	21,7	4,6	36	100
P3	Gravel, dense sand or sand moraine	34	20,0	3,54	9	71
P4	Loose or fine sand, silty sand moraine, silt moraine, loose sand moraine, undrained dry crust $s_u \geq 30$ kPa	30	16,7	3,0	3	50
P5	Loose, well graded sand, wet P3 or P4 under groundwater table, clay and silt s_u = 25-30 kPa	30	12,0	3,0	3	36
P6	Clay and silt s_u = 15-25 kPa	Design is done with both equations for cohesive and frictional soil. The smaller H_{SALL} is chosen.				26
P7	Peat and soft clay	Design is done case by case.				

3.2 Experimental Results

From the recorded Force-Displacement data, the following key parameters are extracted and presented in Section 3.3:

F20 – Lateral load required in order to reach a displacement of 20 mm at the top of the pile at a height of approximately 1.0 m above the ground level. It means that the lateral movement at the ground level is something between 10 mm to 17 mm depending on the location of the centre of rotation in the case of a stiff pile (Fig. 8) or on the location of the hinge point in the case of a bending pile (Fig. 9).

F200 – Lateral load required in order to reach a displacement of 200 mm at the top of the pile at a height of approximately 1.0 m above the ground level. It means that the lateral movement at the ground level was something between 100 mm to 170 mm depending on the location of the centre of rotation in the case of a stiff pile (Fig. 8) or on the location of the hinge point in the case of a bending pile (Fig. 9).

For example, F20 could be used as a criterion for the maximum acceptable movement when designing a foundation for a noise barrier or lighting column. On the other hand, F200 could be a useful parameter in assessing the performance of a safety barrier with rigid piles, typically in soils different from vehicle impact barrier testing track.

The following section presents the results from the experimental programme and compares the results with the design lateral load capacity values obtained from theoretical approaches as described above (Eqns. 1-4).

3.3 Allowed lateral pulling force

The theoretical calculations to define H_{sall} by applying Equations 2, 3 and 4 have been done for each pile type, soil type, embedment depth with flat and sloping ground surfaces. The results of these calculations are presented in Tables 5a, 5b and 5c. The sloping ground conditions for the sloping ground have been derived from the results of the flat ground by multiplying it by 0,42. This value is presented by Kulman (2001) for 1:2 slope case. The material parameters refer Table 4. The material parameters for loose and wet crushed rock (P2) have been derived from the Table 4. The dimensioning values are marked bold in Tables 5a-c. For short piles and sloping ground structures, Equation 2 (soil fails) gives smaller values. Only in case of 3 metre embedment depth, Equation 3 gives smaller values (pile fails) compared to Equation 2, but in these cases Equation 4 limits the design load.

Table 5a. The analytical calculation results of H_{sall} with Equations 2-4 for fragmented rock (F=1).

Soil type (Table 4)	Pile type	Embedment depth m	Ground profile	Eq.2, kN	Eq.3. kN	Eq.4, kN
P1	O-type, wall 8 mm	1	Flat	**5,4**	44,4	14,4
P1	O-type, wall 8 mm	2	Flat	43,7	44,3	**14,6**
P1	HEB	1	Flat	**5,6**	65,7	23,2
P1	O-type, wall 8 mm	1	Slope 1:2	**2,3**	18,7	6,1
P1	O-type, wall 8 mm	2	Slope 1:2	18,4	18,6	**6,1**
P1	HEB	1	Slope 1:2	**2,3**	27,6	9,7

Table 5b. The analytical calculation results of H_{sall} with Equations 2 -4 for moraine (F=1).

Soil type (Table 4)	Pile type	Embedment depth m	Ground profile	Eq.2, kN	Eq.3. kN	Eq.4, kN
P3	O-type, wall 8 mm	1	Flat	**2,5**	32,1	11,0
P3	O-type, wall 8 mm	2	Flat	21,9	31,3	**11,8**
P3	HEB	1	Flat	**2,7**	47,1	18,0

Table 5c. The analytical calculation results of H_{sall} with Equations 2-4 for crushed rock (F=1).

Soil type (Table 4)	Pile type	Embedment depth m	Ground profile	Eq.2, kN	Eq.3. kN	Eq.4, kN
P2	O-type, wall 8 mm	1	Flat	**3,3**	39,3	11,1
P2	O-type, wall 8 mm	2	Flat	26,9	38,9	**11,5**
P2	O-type, wall 8 mm	3	Flat	86,1	39,6	**10,9**
P2	O-type, wall 6,3 mm	3	Flat	86,1	40,1	**9,7**
P2 Dry and wet	HEB	1,2	Flat	**6,1**	57,5	19,8
P2, loose*	HEB	1,2	Flat	**4,3**	48,1	17,8
P2 wet, loose*	HEB	1,2	Flat	**4,3**	48,1	17,8
P2, loose change in grading**	HEB	1,2	Flat	**4,3**	48,1	17,8

*The n_h for wet and loose was chosen to be 25MN/m^3 and $K_{p\gamma}$ 70 kN/m^3.
**The changes in grading are small, P2 loose material parameters were used.

3.4 Analysis of Results and Conclusions

A systematic presentation of results is followed in order to maintain the consistency. The following line-types, colours and bullets are adapted throughout the results (Table 6).

Table 6. Line colour, type and bullets representing soil types, wet-dry conditions, and the pile-type and embedment length, respectively.

Line / Bullet type	Description
	Moraine type soil
	Fragmented rock
	Fragmented rock, slope 1:2
	Crushed aggregate – Barrier Testing Area
– – – – – – –	Wet Crushed aggregate – Barrier Testing Area
	Crushed aggregate – Support Testing Area
– – – – – – –	Wet Crushed aggregate – Support Testing Area
■	H-profile
▲	O-profile with 1 m embedment depth
●	O-profile with 2 m embedment depth
◆	O-profile with 3 m embedment depth

3.4.1 Task 1: Average resistance values (F20, F200, F_{max}) in different soils

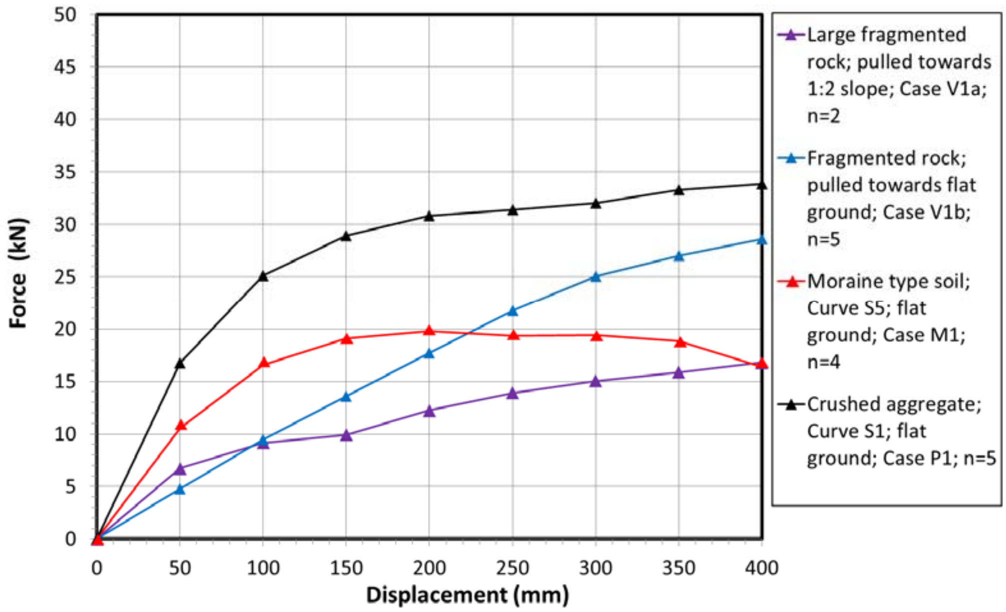


Figure 10. Average force-displacement of O-profile; z=1m in different soils.

Conclusions:

- Crushed aggregate gave the best F20 force, moraine was the second and fragmented rock (with or without slope) was the third
- In Fragmented rock, after 100 mm displacement, the resistance of the pile pulled towards 1:2 slope stayed lower than that of the pile towards on a flat Fragmented rock.
- Crushed aggregate gave the best F200 force. Moraine and fragmented rock were the second with nearly equal F200, if the case with a slope was not taken into account.
- The corresponding design values for H_{sall} from Equation 2 are 5,4 kN for fragmented rock, 2,5 kN for moraine and 3,3 kN for crushed aggregate. These values are close to the F_{20} values.

Table 7. Average force-displacement parameters of O-profile in different soils; z=1m

Soil Type	z (m)	Pull direction Towards ...	Case	F_{20} (kN)	F_{200} (kN)	F_{max} (kN)	S_{Fmax} (mm)
Fragmented rock (Larger than usual)	1	1:2 slope	V1a	3,3	12,2	18,8	492,8
Fragmented rock	1	a flat ground	V1b	2,5	17,7	31,4	457,8
Moraine type – Curve S5	1	a flat ground	M1	4,8	19,8	19,8	210,9
Crushed aggregate-Curve S1	1	a flat ground	P1	6,3	30,8	34,6	439,8

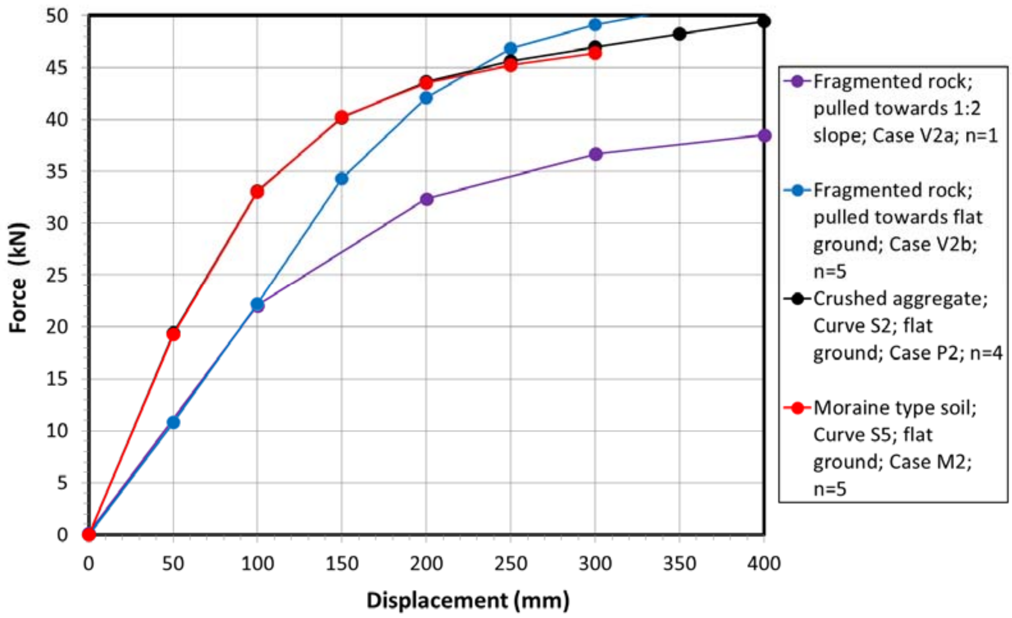


Figure 11. Average force-displacement of O-profile; z =2m in different soils.

Conclusions:

- Crushed aggregate and moraine gave the best F20 force values (almost identical) and followed by the fragmented rock.
- Crushed aggregate and fragmented rock on a flat ground gave the best F200 values and fragmented rock with a slope of 1:2 gave the lowest.
- The corresponding design values for H_{sall} from Equation 4 are 14,6 kN for fragmented rock, 11,8 kN for moraine and 11,5 kN for crushed aggregate. These values are clearly larger than F_{20} values.

Table 8. Average force-displacement parameters of O-profile in different soils; z=2m.

Soil Type	z (m)	Pull direction Towards ...	Case	F_{20} (kN)	F_{200} (kN)	F_{max} (kN)	S_{Fmax} (mm)
Fragmented rock	2	1:2 slope	V2a	4,9	32,4	42,2	564,1
Fragmented rock	2	a flat ground	V2b	4,3	42,1	54,7	429,4
Crushed aggregate-Curve S1	2	a flat ground	P2	7,8	43,7	51,1	458,3
Moraine type – Curve S5	2	a flat ground	M2	7,7	43,5	47,0	342,0

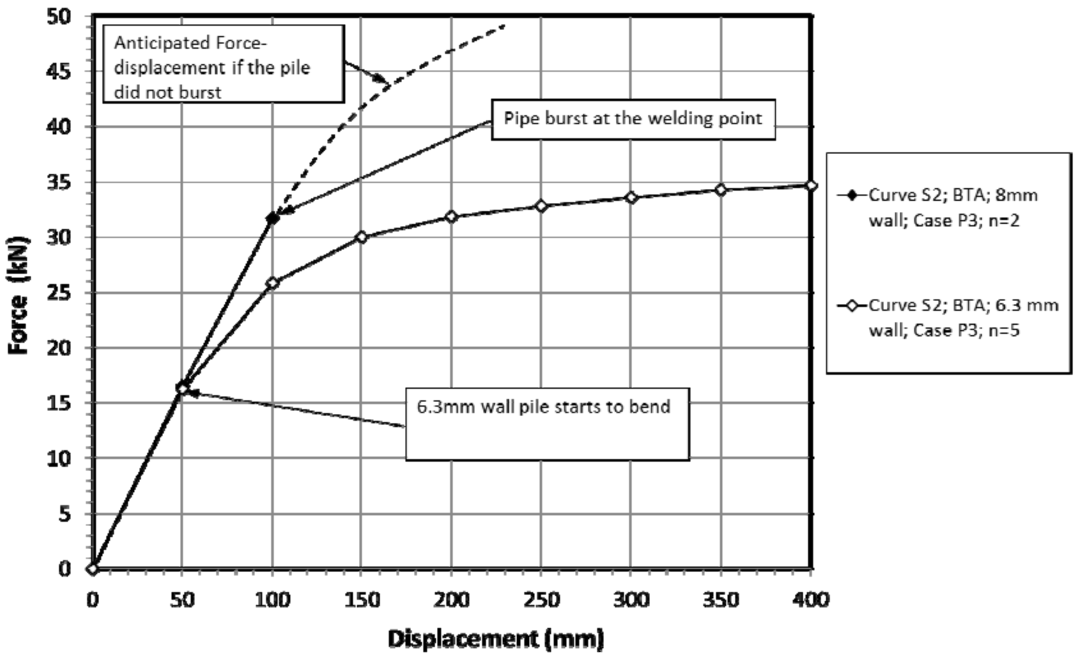


Figure 12. Average force-displacement of O-profile with different wall thickness in flat crushed aggregate ground; z=3m.

Conclusions:

- The wall thickness did not have significant influence at forces lower than displacement of F50; but after 50 mm displacement, the pile with 6.3 mm wall started to develop a plastic hinge.
- At 100 mm, the pile with 8 mm wall burst on welding (at ground level) and the pile with 6.3 mm continued to develop a plastic hinge. The 8mm wall pile was made by welding two separate sections with a length of 1 m and 3 m together. On the other hand, the 6.3 mm wall pile was a single-piece pile.
- The corresponding design values for H_{sall} from Equation 4 are 10,9 kN for crushed aggregate and 8 mm wall thickness, and 9,7 kN for wall thickness 6,3 mm. These values are larger than F_{20} values.

Table 9. Average force-displacement parameters of O-profile in flat crushed aggregate ground with different wall thickness; z=3m.

Soil Type	z (m)	Pull direction Towards ...	Case	F_{20} (kN)	F_{200} (kN)	F_{max} (kN)	S_{Fmax} (mm)
Crushed aggregate-Curve S2 (8mm)	3	a flat ground	P3	5,8	34,4	38,5	198,8
Crushed aggregate –Curve S2 (6.3mm)	3	a flat ground	P3	6,9	31,9	35,2	450,1

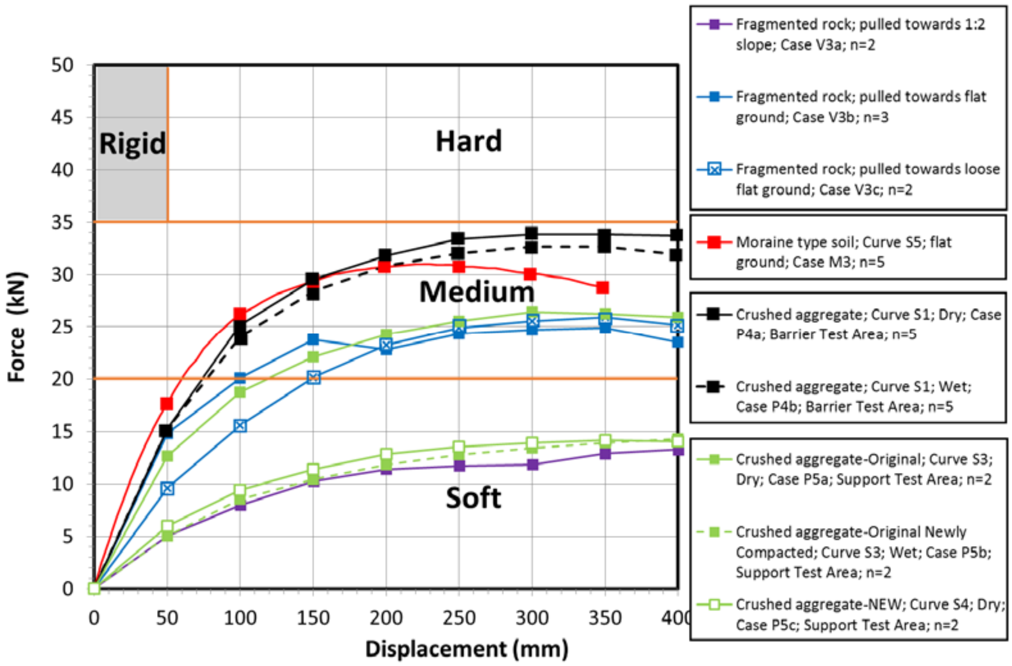


Figure 13. *Average force-displacement of H-profile; z=1m in different soils.*

Conclusions:

- At the Barrier Testing Area (S1), the water content did not affect the performance
- In the Support Testing Area (Curve S3), the newly compacted original aggregate, which was subjected to wetting performed weaker than that of the very same material in its natural (dry) conditions. Considering the observations from Barrier Testing Area where the wet conditions did not have any influence on the pile behaviour, it can be concluded that the poor compaction could have lead to a weaker performance in the newly compacted original aggregate.
- Additionally, the new crushed aggregate that had similar particle size distribution (S4) to the original crushed aggregates (S3) also performed significantly weaker than the original aggregate. This could also be attributable to poor compaction effort (by vibratory plate compactor) during refilling of the Support Testing pit area.
- Interestingly, the performance in moraine type soil was comparable to the performance in crushed aggregate (Curve S1).
- In Fragmented rock, pulling towards a 1:2 slope behaved significantly weaker than pulling towards a flat ground.
- The corresponding design values for H_{sall} from Equation 4 are 5,6 kN and 2,3 for crushed aggregate and flat or sloped ground, respectively. For moraine H_{sall} is 2,7 kN, for dry well compacted crushed aggregate 6,1 kN and for other cases 4,3 kN. In general, these H_{sall} values are smaller than F_{20} values.

Table 10. Average force-displacement parameters of H-profile in different soils; z=1m.

Soil Type	z (m)	Pull direction Towards ...	Case	F_{20} (kN)	F_{200} (kN)	F_{max} (kN)	S_{Fmax} (mm)
Fragmented rock	1	1:2 slope	V3a	2,7	11,4	14,1	347,9
Fragmented rock	1	a flat ground	V3b	8,6	22,8	28,0	279,8
Fragmented rock- loose	1	a flat ground	V3c	4,5	23,2	28,3	589,9
Moraine type – S5	1	a flat ground	M3	8,5	30,8	31,4	228,0
Crushed aggregate- Curve -S1; DRY; BTA	1	a flat ground	P4a	5,9	31,8	33,9	342,1
Crushed aggregate- Curve -S1; WET; BTA	1	a flat ground	P4b	6,4	30,8	34,1	549,8
Crushed aggregate-ORIGINAL: Curve -S3; DRY; STA	1	a flat ground	P5a	6,3	24,2	26,5	507,3
Crushed aggregate-ORIGINAL Newly Compacted: Curve - S3; WET; STA	1	a flat ground	P5b	1,8	11,8	14,6	652,8
Crushed aggregate-NEW: Curve -S4; DRY; STA	1	a flat ground	P5c	2,3	12,8	14,7	589,2

* BTA- Barrier Testing Area; STA – Support Testing Area

3.4.2 Task 2: H- Profile vs O-Profile within 1m depth

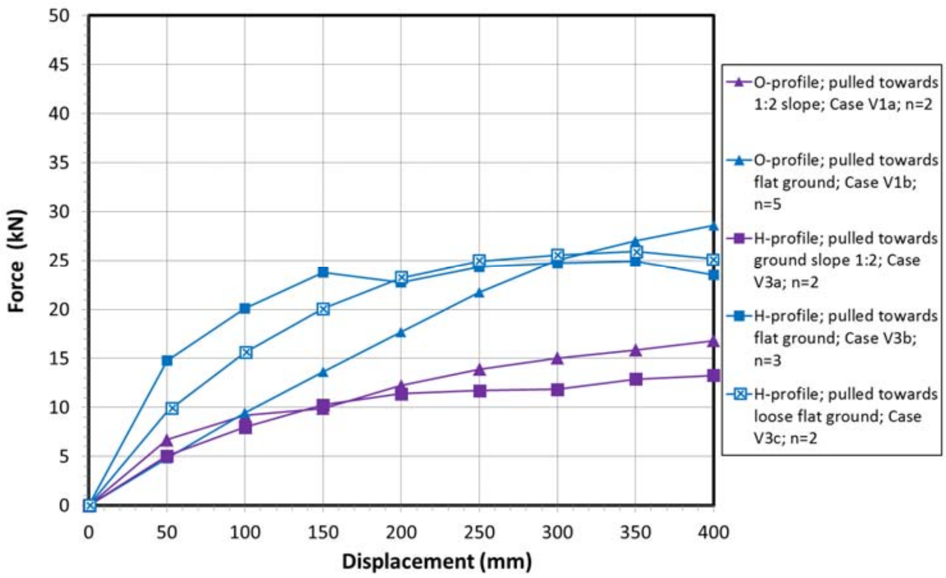


Figure 14. Average force-displacement of H-profile vs O-profile at z=1 m in fragmented rock.

Conclusions:

- Both H- and O-profiles performed weak when pulled towards a 1:2 slope compared to when pulled towards a flat ground.
- H-profiles pulled towards a flat ground gave the best F20. The O-profile and the slope cases with H-profile gave lower resistance. This may be because of the moment of inertia (G) of the H-profile (864 cm^4), which is greater than that of the O-profile (379 cm^4) in the pulling direction.
- After a displacement of 200 mm, the resistance of O-profile is almost identical to the one of H-profile though its inertia was much smaller. Fragmented rock is very heterogenous material and conditions (lots of voids, big particle size) differ significantly. This could lead to differences in installation and eventually to substantial discrepancies in pile behaviour.

Table 11. Average force-displacement parameters of H and O-profiles in Fragmented rock; z=1m.

Soil Type	z (m)	Pull direction Towards …	Case	F_{20} (kN)	F_{200} (kN)	F_{max} (kN)	S_{Fmax} (mm)
Fragmented rock (Larger than usual)	1	1:2 slope	V1a	3,3	12,2	18,8	492,8
Fragmented rock	1	a flat ground	V1b	2,5	17,7	31,4	457,8
Fragmented rock	1	1:2 slope	V3a	2,7	11,4	14,1	347,9
Fragmented rock	1	a flat ground	V3b	8,6	22,8	28,0	279,8
Fragmented rock- loose	1	a flat ground	V3c	4,5	23,2	28,3	589,9

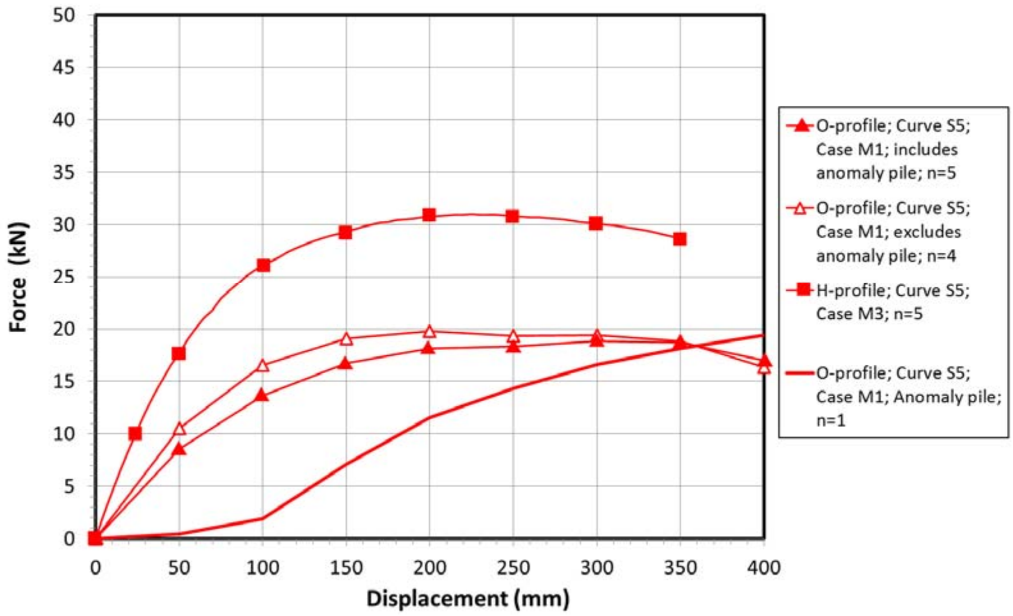


Figure 15. Average force-displacement of H-profile vs O-profile at z=1 m in a flat Moraine type soil.

Conclusions:

- An anomaly pile significantly influenced the average resistance of O-profile piles, thus excluded in the calculations. Possible reason for the anomaly (pile) could be explained as follows; moraine contains different sizes (particle sizes) of rocks. During installation if a pile hits a large piece of rock (d= 30 … 100 mm), it could deviate from its path by displacing the rock away from its origin, creating voids around the pile. These voids would allow the lateral movement of piles even at very small loads as shown in Fig 15.
- In general, H-profile performed better than the O-profile in moraine type soil

Table 12. Average force-displacement parameters of H and O-profiles in Moraine type soil; z=1m.

Soil Type	**z (m)**	**Pull direction Towards ...**	**Case**	**F_{20} (kN)**	**F_{200} (kN)**	**F_{max} (kN)**	**S_{Fmax} (mm)**
O-profile in Moraine type – Curve S5	1	a flat ground	M1	4,8	19,8	19,8	210,9
O-profile in Moraine type – Curve S5	1	a flat ground	M1*	3,9	18,1	21,2	264,5
H-profile in Moraine type – Curve S5	1	a flat ground	M3	8,5	30,8	31,4	228,0

M1*: averages including an anomaly pile

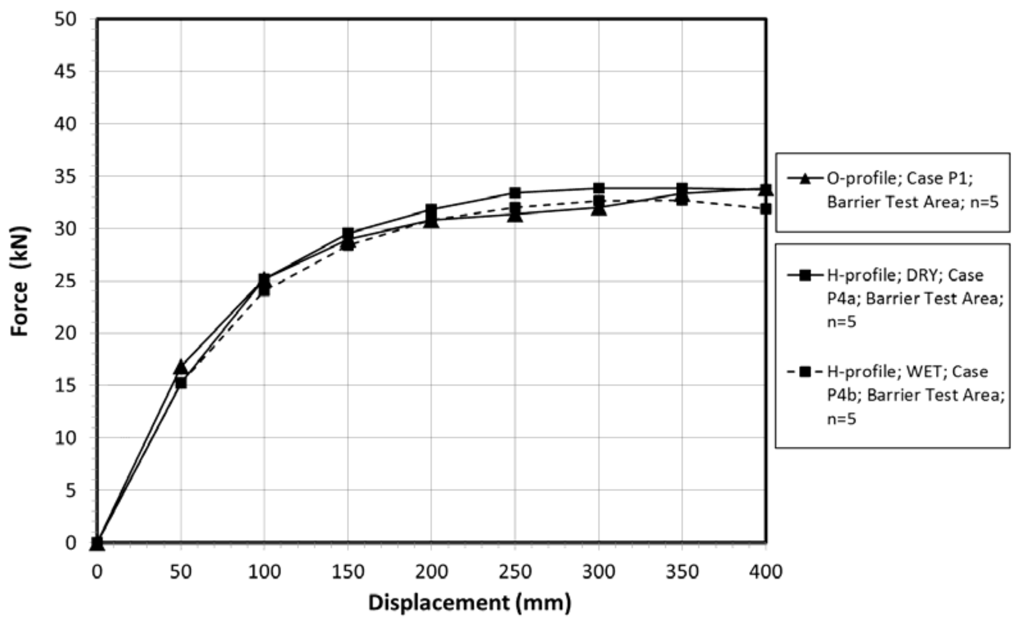


Figure 16. Average force-displacement of H-profile vs O-profile at z=1 m in flat Crushed aggregate: Curve S1 at the Barrier Testing Area.

Conclusions:

- Both H- and O-profiles performed almost identical.
- In the Barrier Testing Area (Curve S1), regardless of the water content of the crushed aggregate, the H-profile performed almost identical in wet and dry conditions.

Table 13. Average force-displacement parameters of H and O-profiles at z=1m in Crushed aggregate: Curve S1 at the Barrier Testing Area.

Soil Type	**z (m)**	**Pull direction Towards ...**	**Case**	**F_{20} (kN)**	**F_{200} (kN)**	**F_{max} (kN)**	**S_{Fmax} (mm)**
O-profile Crushed aggregate – Curve S1	1	a flat ground	P1	6,3	30,8	34,6	439,8
H-profile in Crushed aggregate - Curve S1; DRY; BTA	1	a flat ground	P4a	5,9	31,8	33,9	342,1
H-profile in Crushed aggregate - Curve S1; WET; BTA	1	a flat ground	P4b	6,4	30,8	34,1	549,8

* BTA- Barrier Testing Area

3.4.3 Task 3: Comparison of resistance values with depth within the same soil

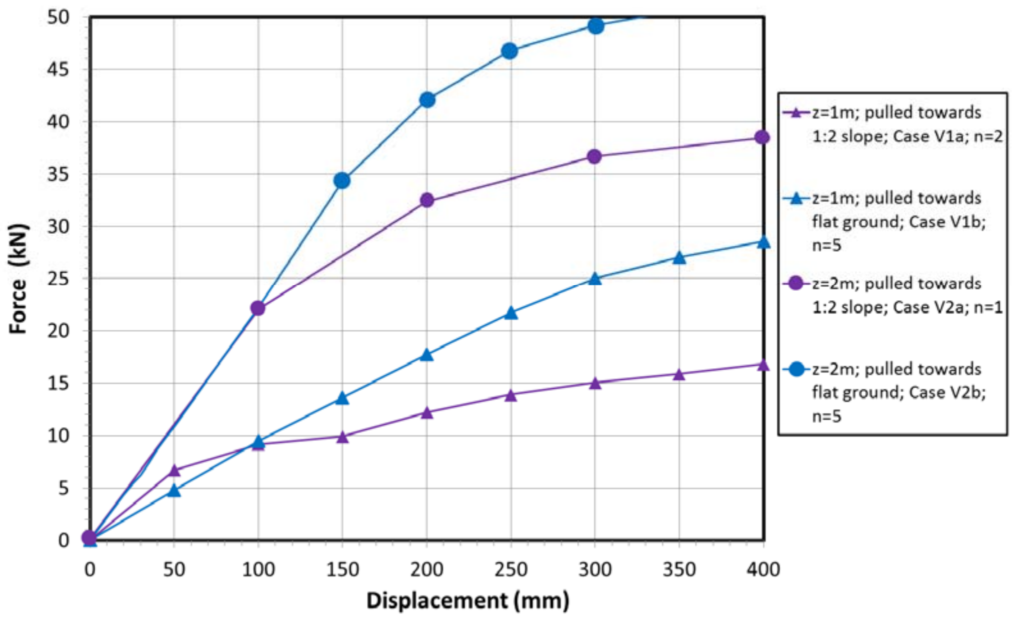


Figure 17. Average force-displacement of O-profiles with different embedment depths in fragmented rock.

Conclusions:

- Piles with z = 2 m gave the highest F20 values compared to the piles with z = 1 m.
- After a displacement of 100 mm, the resistance of piles pulled towards a 1:2 slope stayed lower than that of similar piles but on a flat ground.

Table 14. Average force-displacement parameters of O-profiles with different embedment depths in fragmented rock.

Soil Type	z (m)	Pull direction Towards …	Case	F_{20} (kN)	F_{200} (kN)	F_{max} (kN)	S_{Fmax} (mm)
Fragmented rock (Larger than usual)	1	1:2 slope	V1a	3,3	12,2	18,8	492,8
Fragmented rock	1	a flat ground	V1b	2,5	17,7	31,4	457,8
Fragmented rock	2	1:2 slope	V2a	4,9	32,4	42,2	564,1
Fragmented rock	2	a flat ground	V2b	4,3	42,1	54,7	429,4

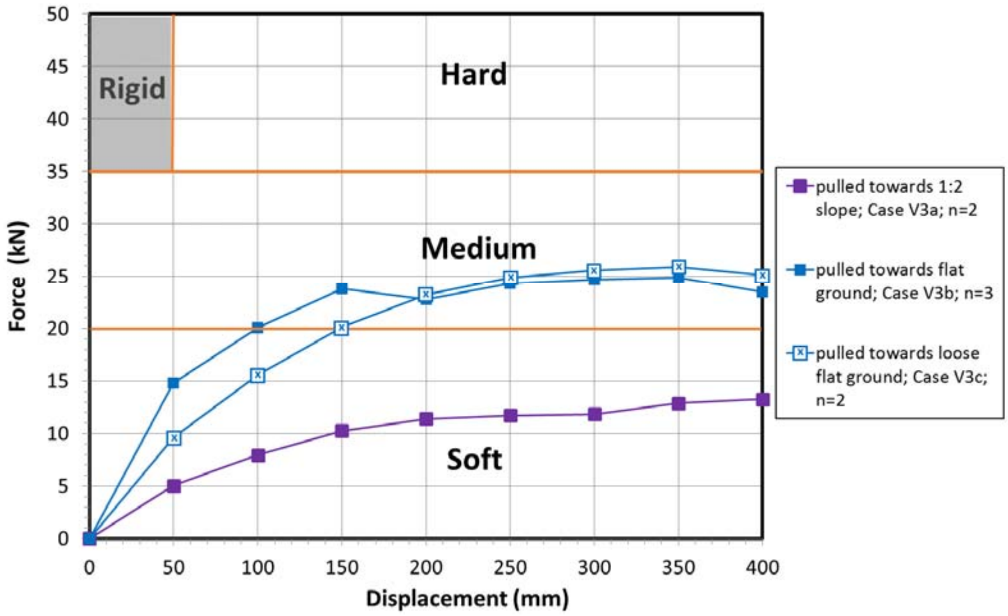


Figure 18. Average force-displacement of H-profiles in fragmented rock: z=1m

Conclusions:

- Pulling towards a 1:2 slope resulted in the weakest performance of H-profile in fragmented rock
- On a flat ground, especially at the beginning, some variation in pile behaviour between the loose and regular part of the fragmented rock was observed. This could be attributable to the non-uniform thickness of the crushed aggregate layer that was placed on top of the fragmented rock. It is typical to observe such variation at the beginning of loading where the pile has less resistance from the surrounding soil.

Table 15. Average force-displacement parameters of H-profiles in fragmented rock: z=1m

Soil Type	z (m)	Pull direction Towards ...	Case	F_{20} (kN)	F_{200} (kN)	F_{max} (kN)	S_{Fmax} (mm)
Fragmented rock	1	1:2 slope	V3a	2,7	11,4	14,1	347,9
Fragmented rock	1	a flat ground	V3b	8,6	22,8	28,0	279,8
Fragmented rock- loose	1	a flat ground	V3c	4,5	23,2	28,3	589,9

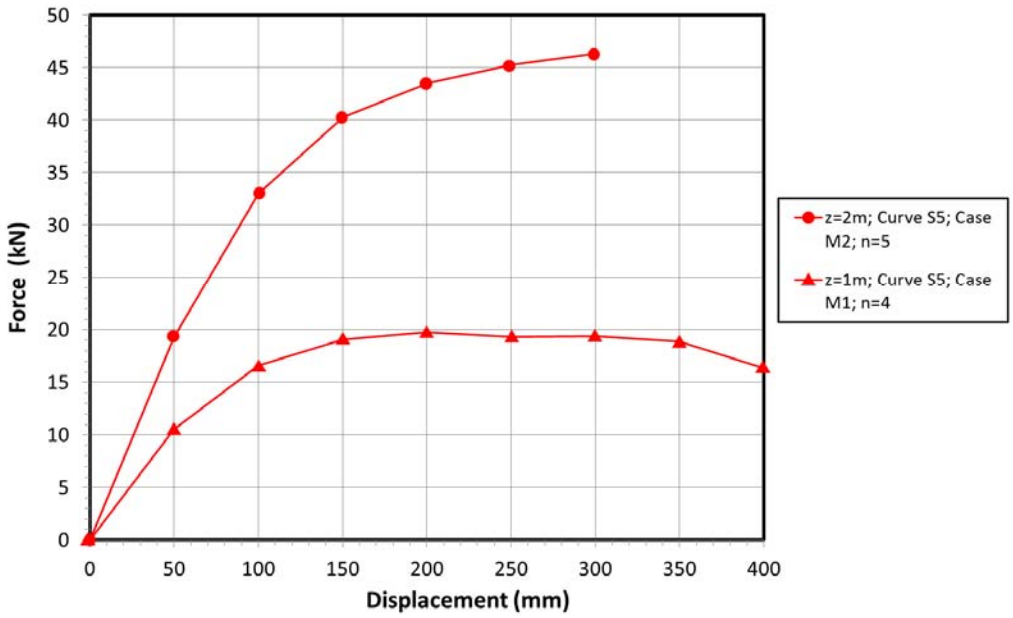


Figure 19. Average force-displacement of O-profiles with different embedment depth in flat moraine type soil

Conclusions:

- The greater the embedment depth, the better the performance is; plies with an embedment depth of 2 m performed better than that of the piles with 1 m embedment depth.

Table 16. Average force-displacement parameters of O-profiles with different embedment depth in flat moraine type soil

Soil Type	z (m)	Pull direction Towards …	Case	F_{20} (kN)	F_{200} (kN)	F_{max} (kN)	S_{Fmax} (mm)
Moraine type – Curve S5	2	a flat ground	M2	7,7	43,5	47,0	342,0
O-profile in Moraine type – Curve S5	1	a flat ground	M1	4,78	19,8	19,8	210,9
O-profile in Moraine type – Curve S5	1	a flat ground	M1*	3,9	18,1	21,2	264,5

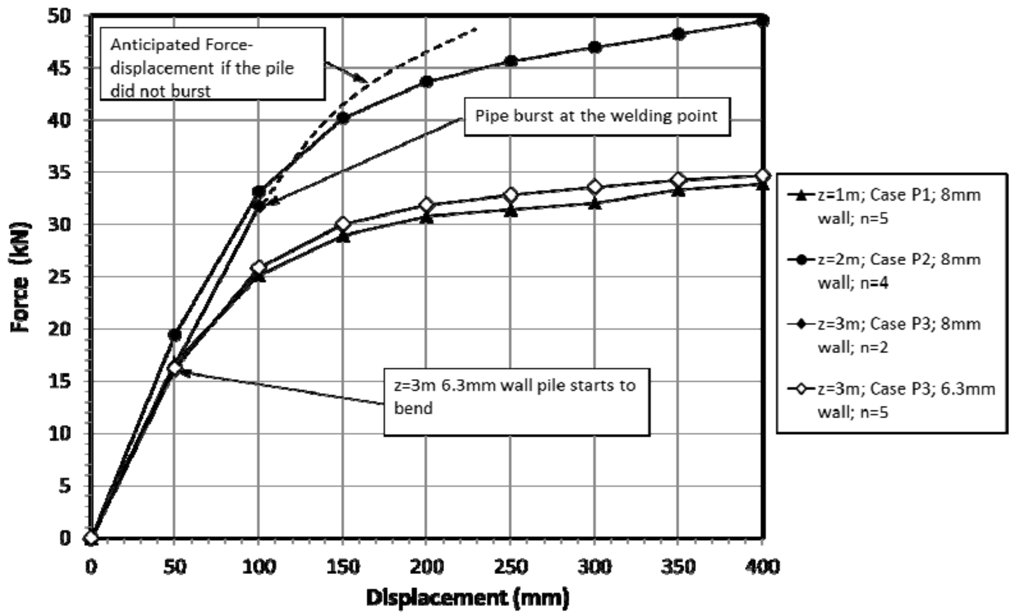


Figure 20. *Average force-displacement of O-profiles with different embedment depths in flat crushed aggregate - Curve S1 at Barrier Testing Area.*

Conclusions:

- Piles with z = 2 m gave a better F20 and F200 values than that of the piles with z = 1 m.
- Piles with z = 3 m were expected to give the highest F20 and F200 values, but they failed to do so. The pile with 6.3 mm wall started to bend (plastic hinge formed) near the ground surface and the pile with 8 mm burst at the welding point.

Table 17. *Average force-displacement parameters of O-profiles with different embedment depths in flat crushed aggregate - Curve S1 at Barrier Testing Area*

Soil Type	z (m)	Pull direction Towards ...	Case	F_{20} (kN)	F_{200} (kN)	F_{max} (kN)	S_{Fmax} (mm)
Crushed aggregate -Curve S1	1	a flat ground	P1	6,3	30,8	34,6	439,8
Crushed aggregate-Curve S1	2	a flat ground	P2	7,8	43,7	51,1	458,3
Crushed aggregate-Curve S2 (8mm)	3	a flat ground	P3	5,8	34,4	38,5	198,8
Crushed aggregate – Curve S2 (6.3mm)	3	a flat ground	P3*	6,9	31,9	35,2	450,1

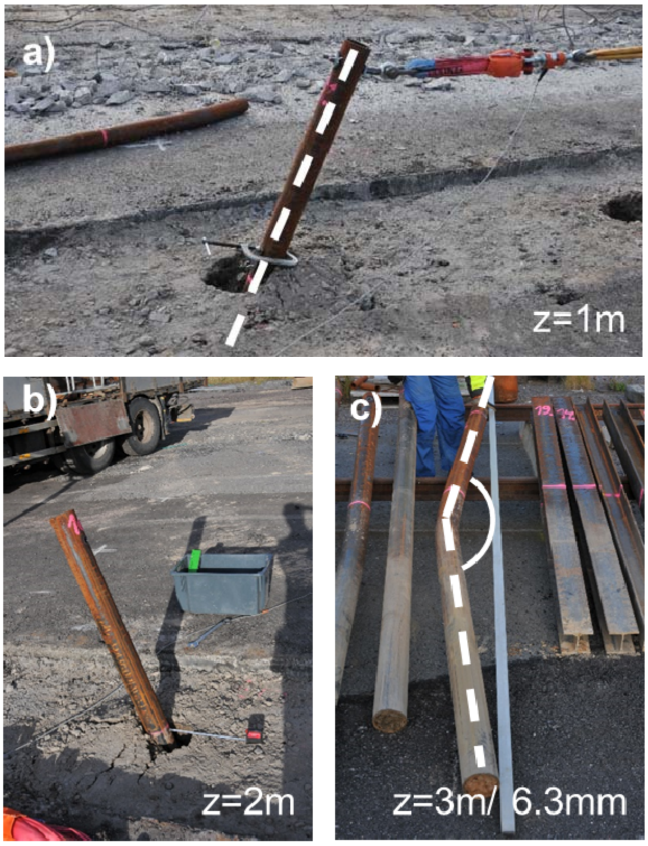


Figure 21. *Field observations of O-profiles with different embedment depths in flat crushed aggregate: Barrier Testing Area - Curve S1: a) z=1m behaved as a rigid pile, b) z=2m behaved similar to z=1m, and c) z=3m with 6.3 mm wall thickness endured permanent deformation- a plastic hinge formed.*

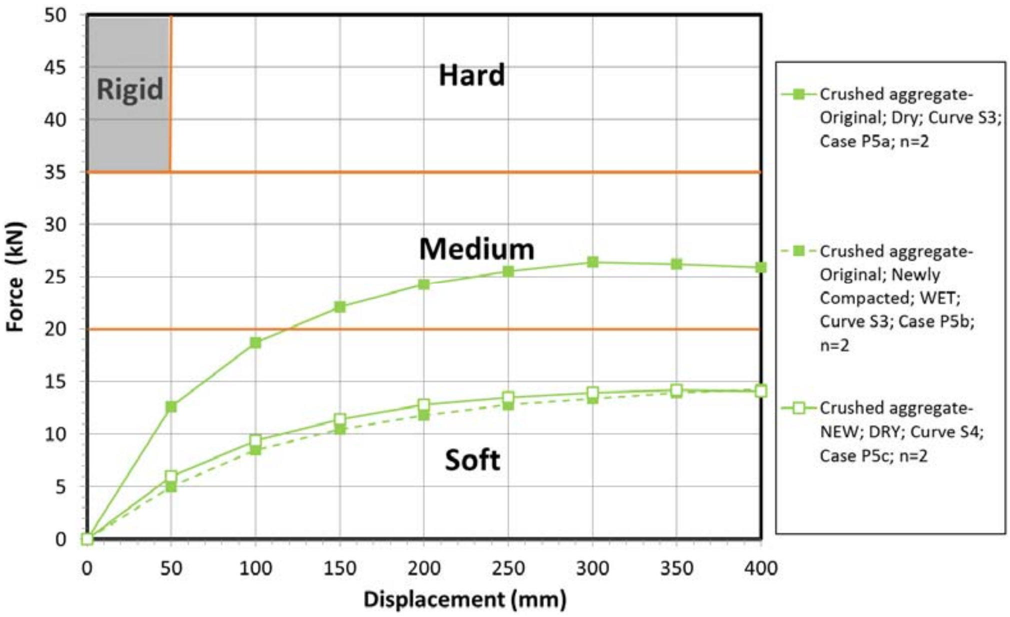


Figure 22. *Average force-displacement of H-profiles in a flat crushed aggregate ground at the Support Testing Area: Curves S3 and S4.*

Conclusions:

- The H-profile in the original crushed aggregate (Curve S3), which was compacted originally with a vibratory plate and was treated with repeated rains during an extended period produced the highest F20 and F200 values. The same soil first removed and then compacted with the same vibratory plate and then wetted gave 30 to 50 % lower results when it was tested immediately after compaction.
- In the third case, the original aggregate was replaced with a new crushed aggregate that had similar grading (S4) to the original crushed aggregates (S3). It was compacted the similar way but not wetted. It gave the same results as the original aggregate just after compaction.

Table 18. Average force-displacement parameters of H-profiles in a flat crushed aggregate ground at the Support Testing Area: Curves S3 and S4.

Soil Type	z (m)	Pull direction Towards ...	Case	F_{20} (kN)	F_{200} (kN)	F_{max} (kN)	S_{Fmax} (mm)
Crushed aggregate-ORIGINAL: Curve S3; DRY; STA	1	a flat ground	P5a	6,3	24,2	26,5	507,3
Crushed aggregate-ORIGINAL Newly Compacted: Curve S3; WET; STA	1	a flat ground	P5b	1,8	11,8	14,6	652,8
Crushed aggregate-NEW: Curve S4; DRY; STA	1	a flat ground	P5c	2,3	12,8	14,7	589,2

*STA- Support Testing Area

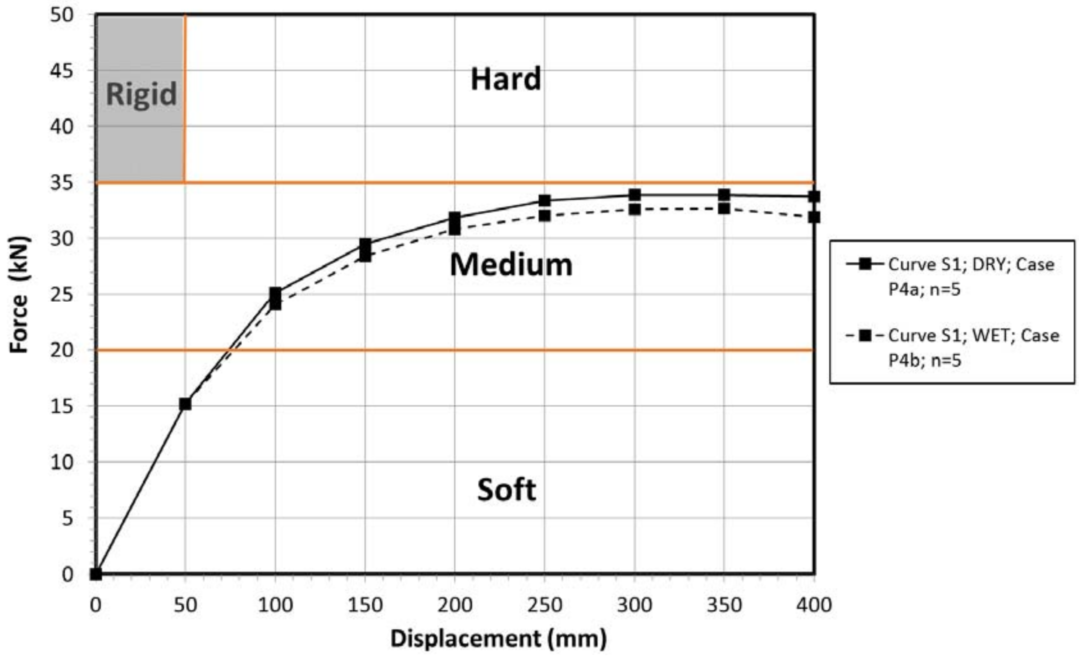


Figure 23. Average force-displacement of H-profiles in dry and wet crushed aggregate- flat ground at the Barrier Testing Area: Curve S1.

Conclusions:

- The water content did not affect the performance of H-profile piles, implying that the crushed aggregate at the Barrier Testing Area was not sensitive to water.

Table 19. Average force-displacement parameters of H-profiles in dry and wet crushed aggregate- flat ground at the Barrier Testing Area: Curve S1.

Soil Type	z (m)	Pull direction Towards ...	Case	F_{20} (kN)	F_{200} (kN)	F_{max} (kN)	S_{Fmax} (mm)
Crushed aggregate-Curve S1;DRY;BTA	1	Towards a flat ground	P4a	5,9	31,8	33,9	342,1
Crushed aggregate-Curve S1;WET;BTA	1	Towards a flat ground	P4b	6,4	30,8	34,1	549,8

*BTA- Barrier Testing Area

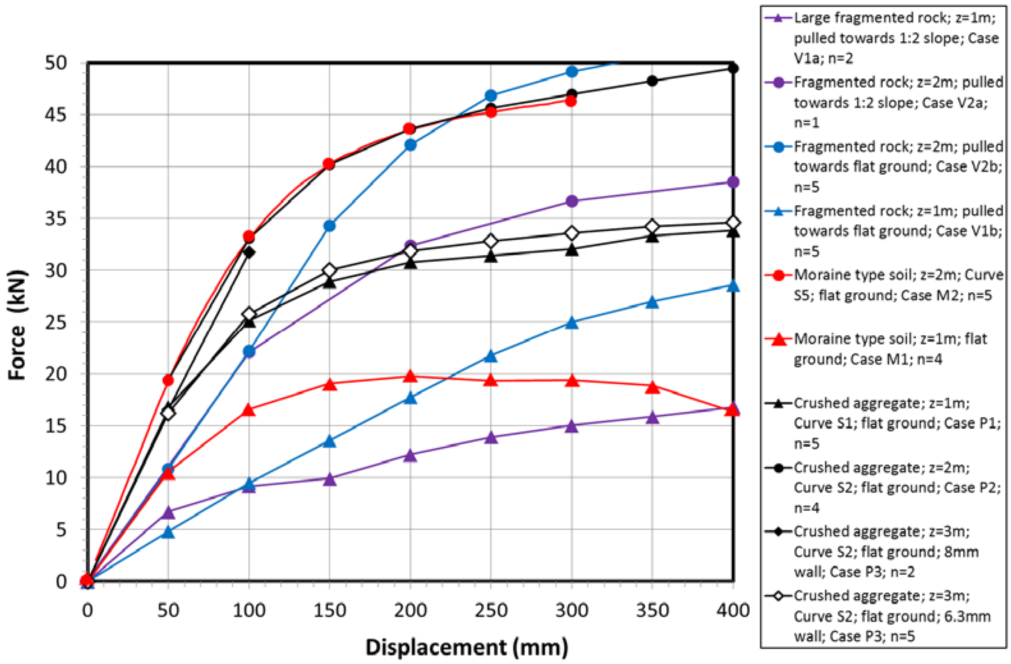


Figure 24. Summary of average force-displacement of all O-profile piles.

Table 20. Summary of average force-displacement parameters of all O-profile piles.

Soil Type	z (m)	Pull direction Towards ...	Case	F_{20} (kN)	F_{200} (kN)	F_{max} (kN)	S_{Fmax} (mm)
Fragmented rock (Larger than usual)	1	1:2 slope	V1a	3,3	12,2	18,8	492,8
Fragmented rock	1	a flat ground	V1b	2,5	17,7	31,4	457,8
Fragmented rock	2	1:2 slope	V2a	4,9	32,4	42,2	564,1
Fragmented rock	2	a flat ground	V2b	4,3	42,1	54,7	429,4
Moraine type – Curve S5	1	a flat ground	M1	4,8	19,8	19,8	210,9
Moraine type – Curve S5	1	a flat ground	M1*	3,9	18,1	21,2	264,5
Moraine type – Curve S5	2	a flat ground	M2	7,7	43,5	47,0	342,0
Crushed aggregate-Curve S1	1	a flat ground	P1	6,3	30,8	34,6	439,8
Crushed aggregate-Curve S1	2	a flat ground	P2	7,8	43,7	51,1	458,3
Crushed aggregate-Curve S2 (8mm)	3	a flat ground	P3	5,8	34,4	38,5	198,8
Crushed aggregate – Curve S2 (6.3mm)	3	a flat ground	P3*	6,9	31,9	35,2	450,1

Table 21a. Summary of average force-displacement parameters of O-profile piles: sorted according to the F20 in descending order.

Soil Type	z (m)	Pull direction Towards ...	Case	F_{20} (kN)	F_{200} (kN)	F_{max} (kN)	S_{Fmax} (mm)
Crushed aggregate-Curve S1	2	a flat ground	P2	7,8	43,7	51,1	458,3
Moraine type – Curve S5	2	a flat ground	M2	7,7	43,5	47,0	342,0
Crushed aggregate-Curve S1	1	a flat ground	P1	6,3	30,8	34,6	439,8
Fragmented rock	2	1:2 slope	V2a	4,9	32,4	42,2	564,1
Moraine type – Curve S5	1	a flat ground	M1	4,8	19,8	19,8	210,9
Fragmented rock	2	a flat ground	V2b	4,3	42,1	54,7	429,4
Fragmented rock (Larger than usual)	1	1:2 slope	V1a	3,3	12,2	18,8	492,8
Fragmented rock	1	a flat ground	V1b	2,5	17,7	31,4	457,8

If F20 is used to analyse the pile behaviour:

- Piles with z=2m in moraine type soil and in crushed aggregate performed the best when pulled towards a flat ground.
- Among the z=1m piles, the one in crushed aggregate performed extremely well compared to others.
- In general, piles in fragmented rock performed the worst. The z=2m piles in fragmented rock performed weaker than the z=1m pile in a flat crushed aggregate ground.
- z=1m pile in moraine performed better than the z=2m pile in flat fragmented rock.
- The values of allowed pulling force H_{sall} for different pile types varied significantly: for short O-profile (z=1m) piles, the H_{sall} matched well with measured values. In contrast, for longer piles (z=2 – 3 m) the H_{sall} was clearly larger than F_{20}.

Table 21b. Summary of average force-displacement parameters of O-profile piles: sorted according to the F200 in descending order.

Soil Type	z (m)	Pull direction Towards …	Case	F_{20} (kN)	F_{200} (kN)	F_{max} (kN)	S_{Fmax} (mm)
Crushed aggregate-Curve S1	2	a flat ground	P2	7,8	43,7	51,1	458,3
Moraine type – Curve S5	2	a flat ground	M2	7,7	43,5	47,0	342,0
Fragmented rock	2	a flat ground	V2b	4,3	42,1	54,7	429,4
Fragmented rock	2	1:2 slope	V2a	4,9	32,4	42,2	564,1
Crushed aggregate-Curve S1	1	a flat ground	P1	6,3	30,8	34,6	439,8
Moraine type – Curve S5	1	a flat ground	M1	4,8	19,8	19,8	210,9
Fragmented rock	1	a flat ground	V1b	2,5	17,7	31,4	457,8
Fragmented rock (Larger than usual)	1	1:2 slope	V1a	3,3	12,2	18,8	492,8

If F200 is used to analyse the pile behaviour:

- Piles with z=2m in Moraine type soil and in Crushed aggregate continued to perform well. Despite its weaker performance in lower loads, the z=2m piles in flat Fragmented rock performed well at F200.
- Piles with z=1m in Fragmented rocks continued to perform the worst followed by the pile with z=1m in Moraine.

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SFS-EN 933-2. Kiviainesten geometristen ominaisuuksien testaus. Osa 2. Rakeisuuden määrittäminen. Seulasarjat, aukkojen nimelliskoko. Suomen standardisoimisliitto 1996.

Appendix 1 H-piles in Crushed Aggregate - Pori

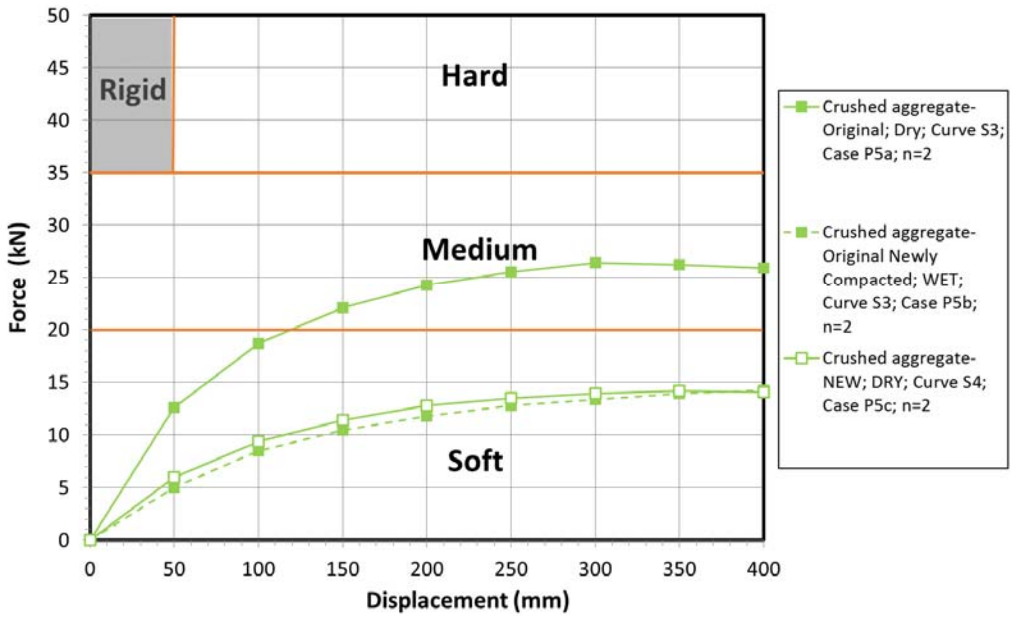


Figure A1. *Average force-displacement of H-profile with different backfill; z= 1 m: Support testing area in Pori*

Table A1. *Average force-displacement parameters of H-profiles with different backfill; z= 1 m: Support testing area in Pori*

Soil Type	z (m)	Pull direction Towards ...	Case	F_{20} (kN)	F_{200} (kN)	F_{max} (kN)	S_{Fmax} (mm)
Crushed aggregate-ORIGINAL: Curve S3; DRY; STA	1	Towards a flat ground	P5a	6,3	24,2	26,5	507,3
Crushed aggregate-ORIGINAL Newly Compacted: Curve S3; WET; STA	1	Towards a flat ground	P5b	1,8	11,8	14,6	652,8
Crushed aggregate-NEW: Curve S4; DRY; STA	1	Towards a flat ground	P5c	2,3	12,8	14,7	589,2

*STA- Support Testing Area

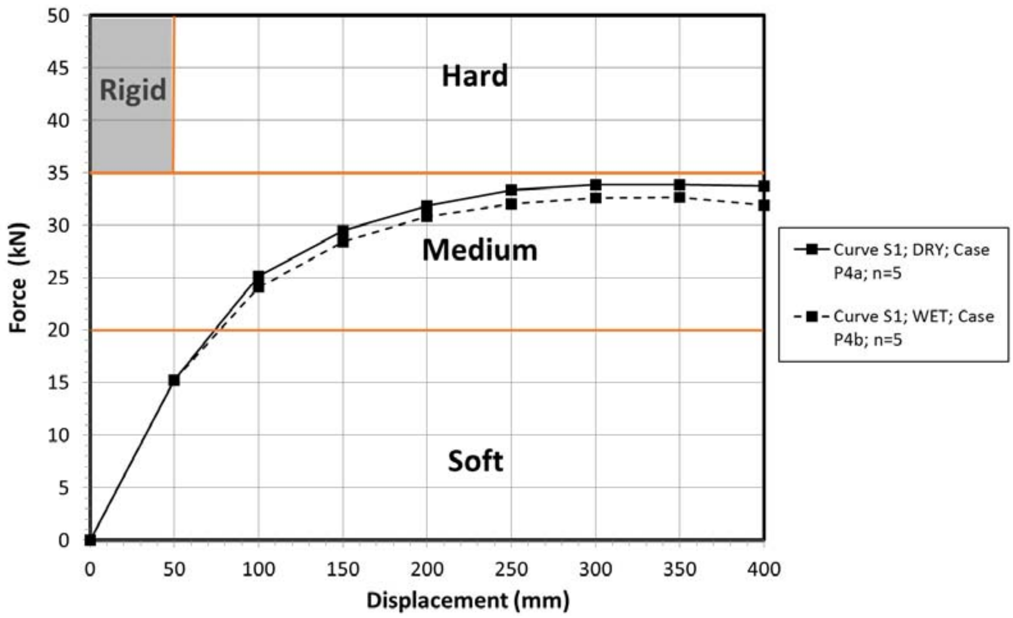
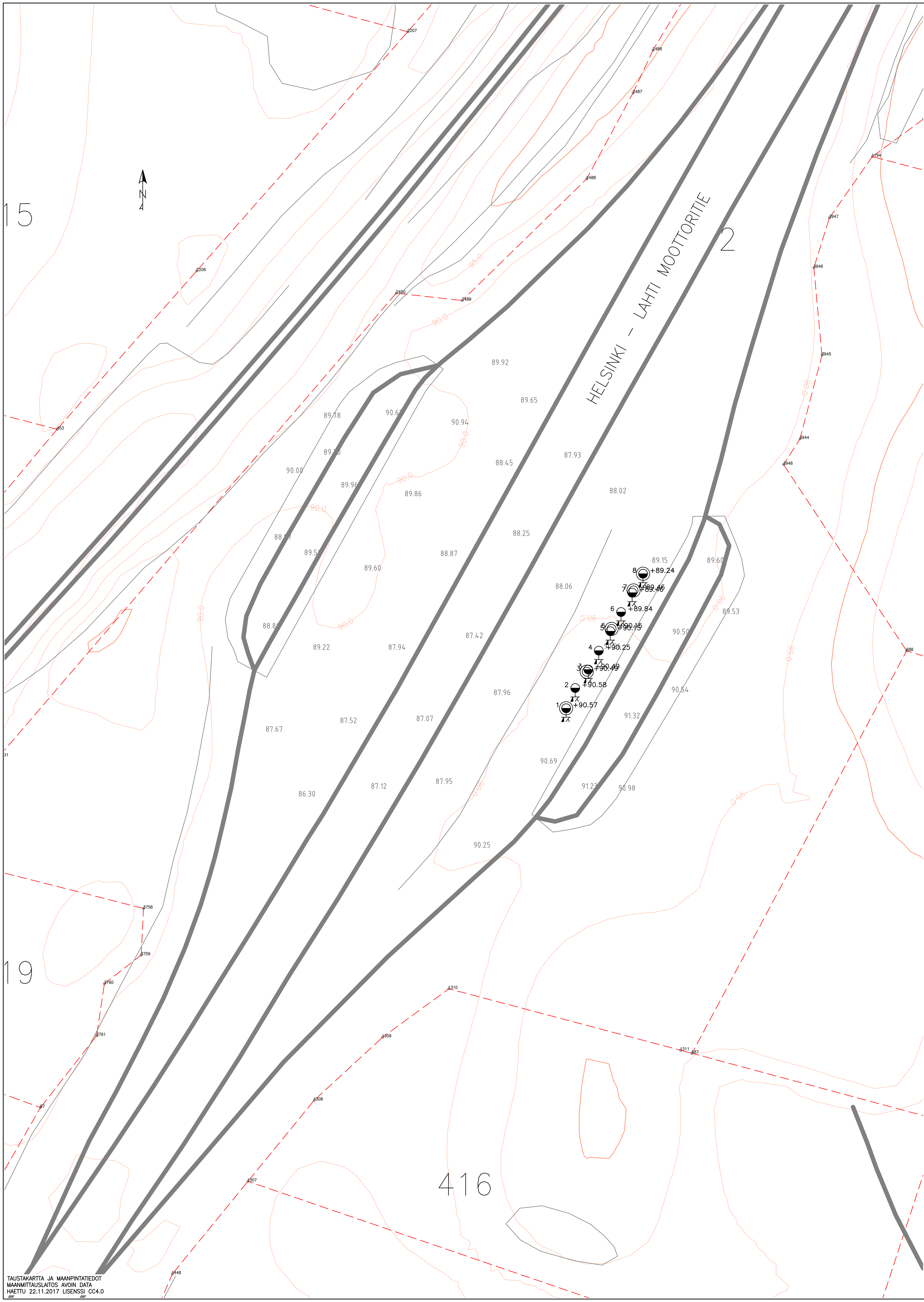


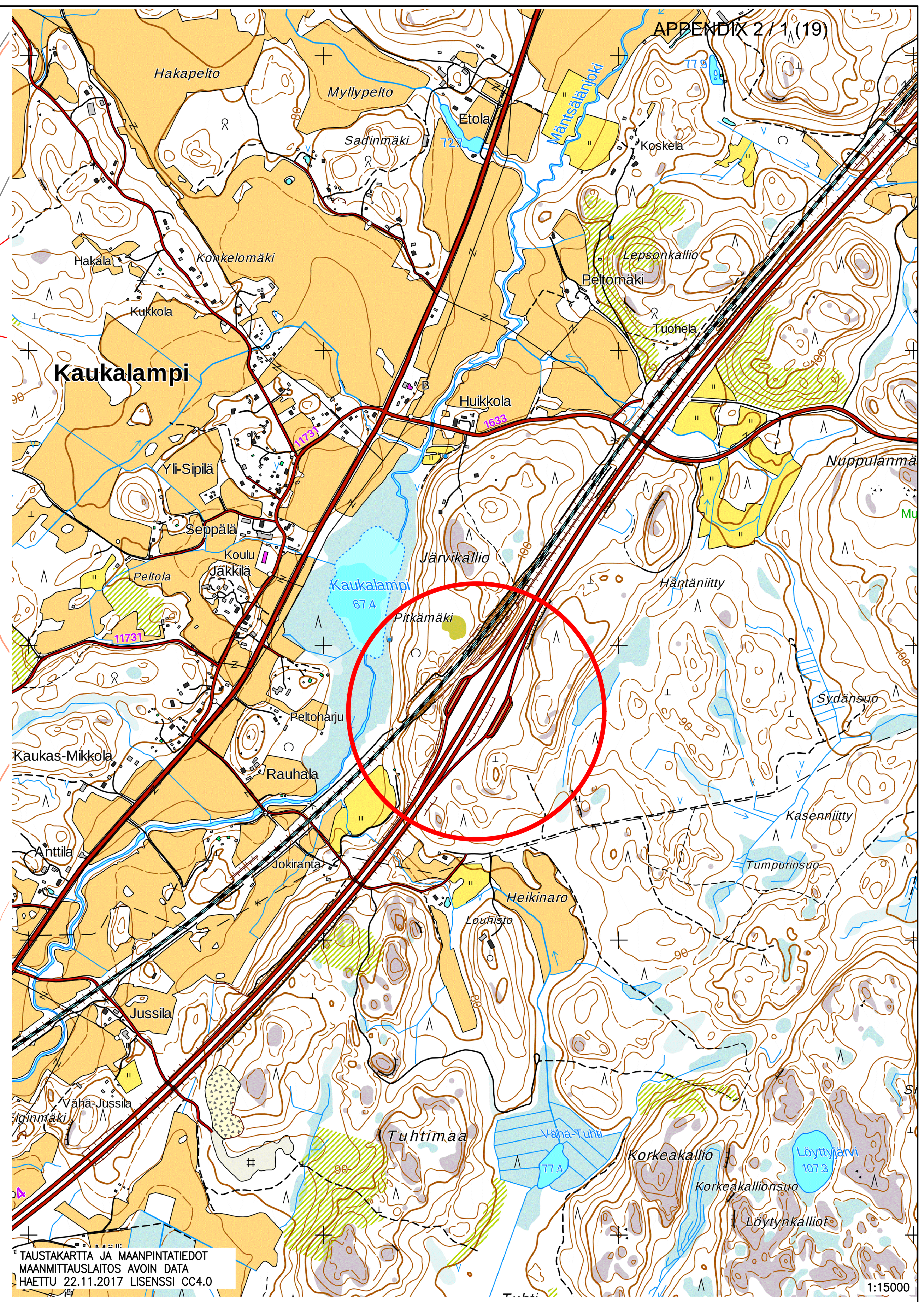
Figure A2. *Average force-displacement of H-profile under dry and wet conditions; z= 1 m: Barrier testing area in Pori*

Table A2. *Average force-displacement parameters of H-profile under dry and wet conditions; z= 1 m: Barrier testing area in Pori*

Soil Type	z (m)	Pull direction Towards …	Case	F_{20} (kN)	F_{200} (kN)	F_{max} (kN)	S_{Fmax} (mm)
Crushed aggregate-Curve S1; DRY; BTA	1	Towards a flat ground	P4a	5,9	31,8	33,9	342,1
Crushed aggregate-Curve S1; WET; BTA	1	Towards a flat ground	P4b	6,4	30,8	34,1	549,8

*BTA- Barrier Testing Area





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GEOPALVELU OY MIKKOLANTIE 11 33470 YLÖJÄRVI P (03)2767200 suunnittelu@geopalvelu.fi			
SUUNNITTELIJA	TUTKIJA TV, MA		SUUNNITTELUALA: GEO TYÖNUMERO: 17230 PIIRUSTUSNUMERO: 202
HYVÄKSYJÄ	PIIRTÄJÄ ML		
KOORDINAATTI– JA KORKEUSJÄRJESTELMÄ ETRS–TM35FIN			PVM 22.11.2017

8.11.2017

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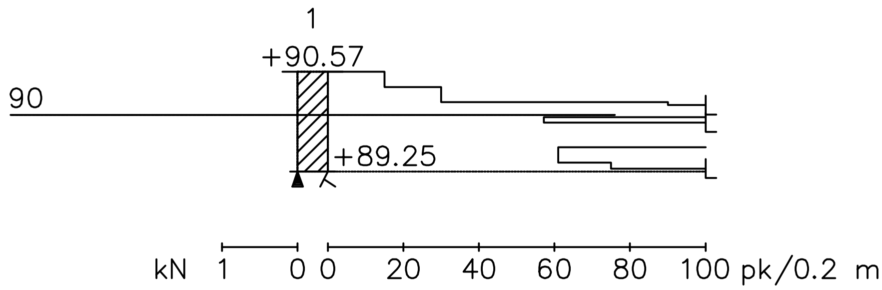

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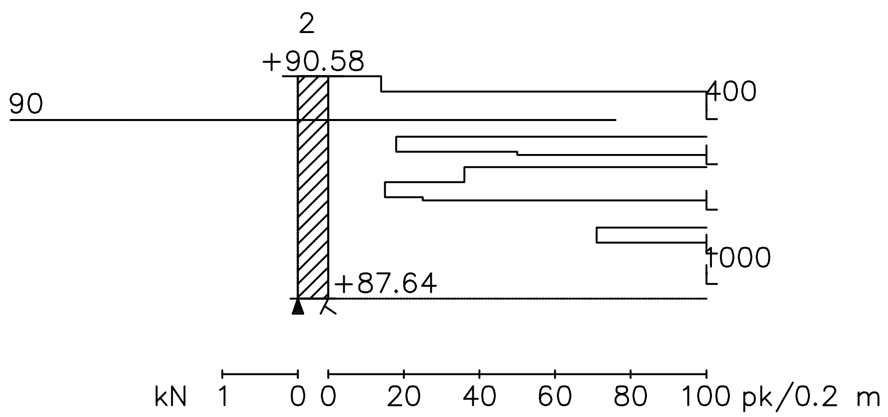

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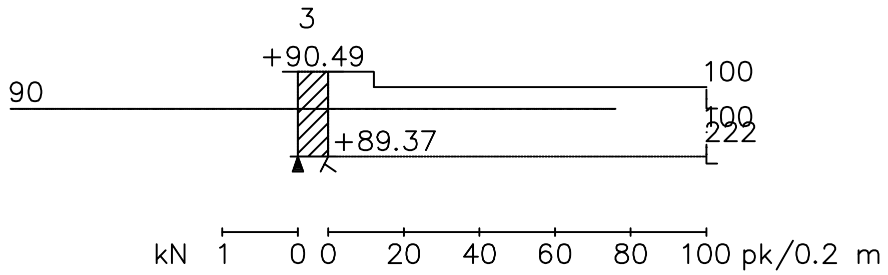

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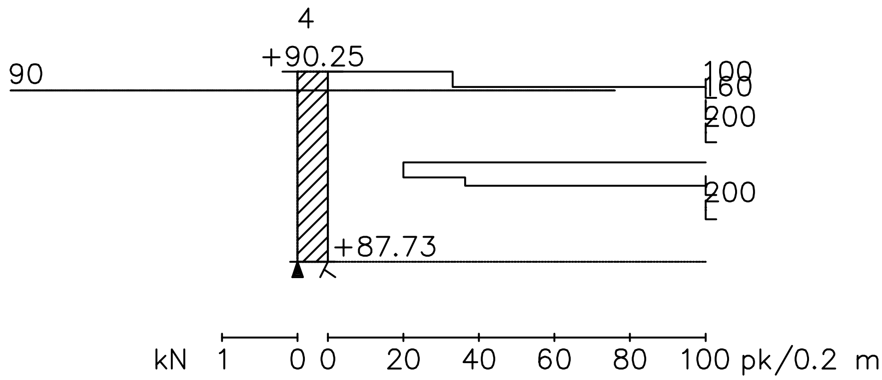

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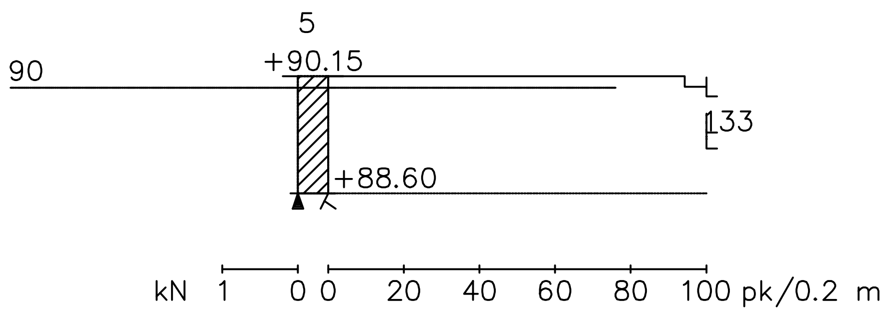

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PA – Painokairaus		Kivi, lohkare tai kallio	
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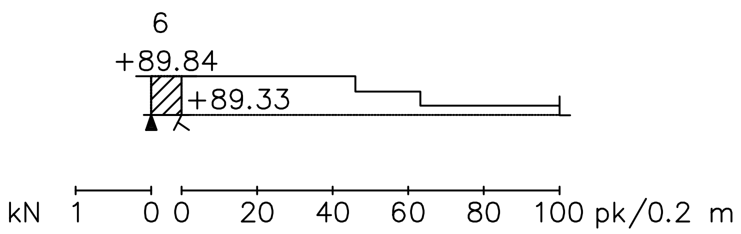

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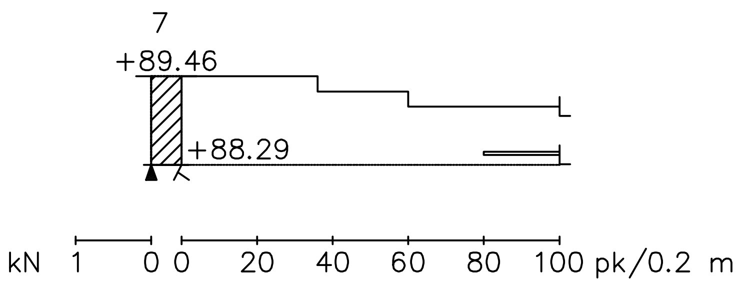

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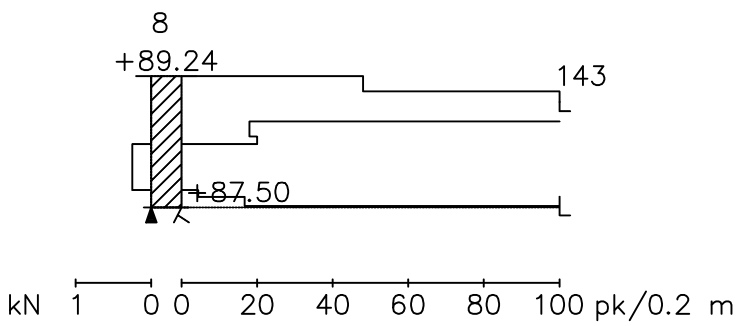
PA – Painokairaus		Kivi, lohkare tai kallio	
Kairaaja		Kairauslaite	



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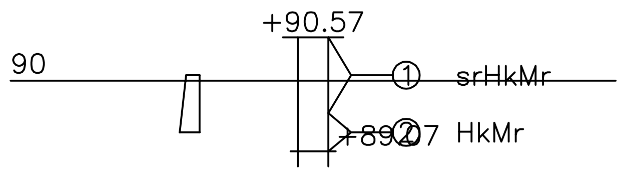

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LABORATORION TUTKIMUSSELOSTUS

Sivu 1
16.11.2017

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Tilaaja	Tutkimus

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Paalu					
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Korkeustaso	90.07	89.32			
Ottoaika	14.11.2017	14.11.2017			
Irtotiheys: kuiva, märkä					
Kiintotiheys					
Vesipitoisuus %	8.9	13.3			
Humus: poltto, NaOH %					
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Kantavuusluokka					
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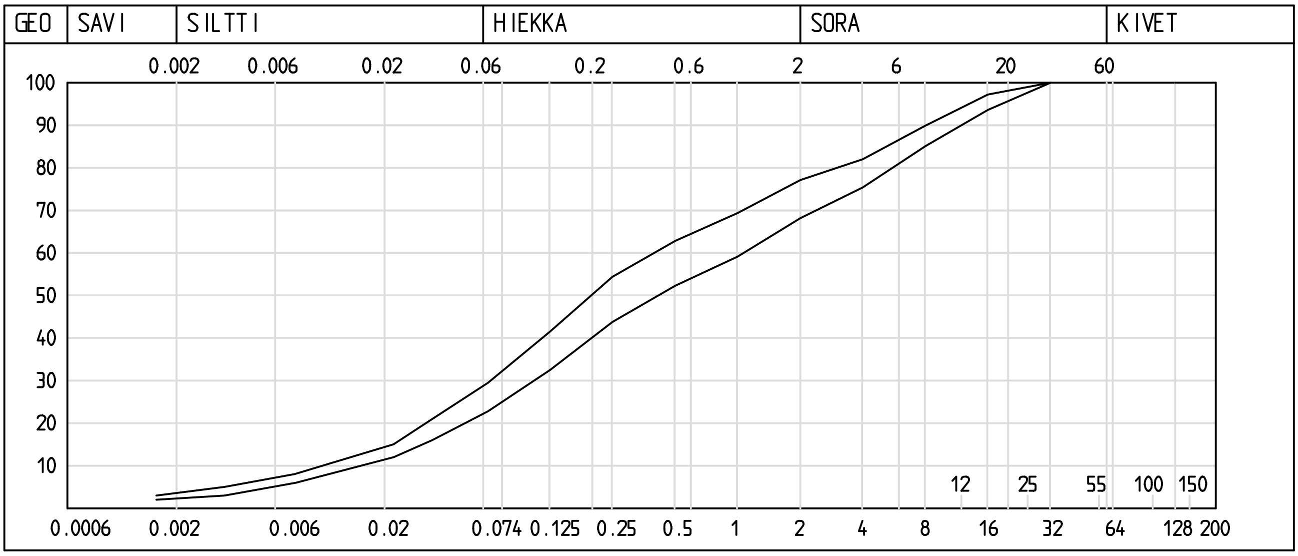

Lausunto

16.11.2017

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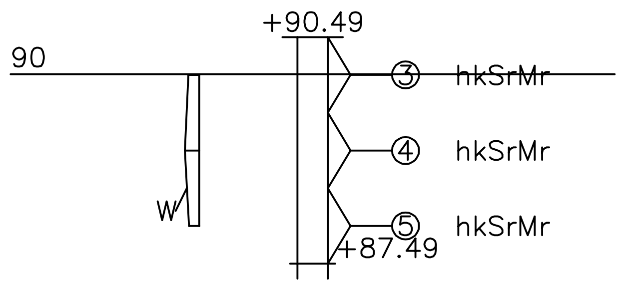
Korkeusjärjestelmä	Pohjaveden pinta	Kairauspvm.	Alkukairaus
N2000		14.11.2017	
Kairaustapa		Päättymistapa	
NĂ - Häiriintynyt näyte		Määräsyvyys	
Kairaaja		Kairauslaite	



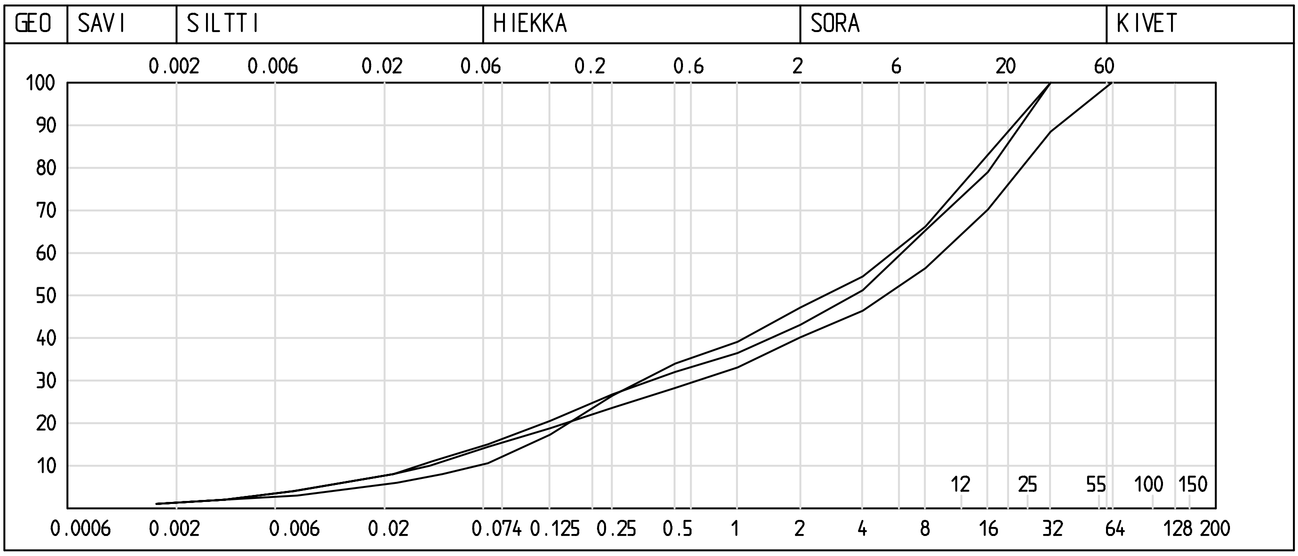
Mittakaava 1:100

LABORATORION TUTKIMUSSELOSTUS

Sivu 1
16.11.2017

Karttalehti	Pisteen nimi MANTSALA	Pisteen nro 3	Työnumero 17230
X 6729756.914	Y 412570.004	Z 90.490	
Arkistonumero	Suunnitelmanumero		
Tilaaja		Tutkimus	

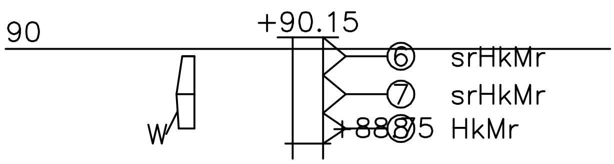
Näytteen tunnus	a	b	c		
Laboratorionumero	3/N0864022	4/N0864023	5/N0864024		
Paalu					
Syvyys	0.50	1.50	2.50		
Korkeustaso	89.99	88.99	87.99		
Ottoaika	14.11.2017	14.11.2017	14.11.2017		
Irtotiheys: kuiva, märkä					
Kiintotiheys					
Vesipitoisuus %	7.3	9.6	6.9		
Humus: poltto, NaOH %					
Routivuus: routimaton, routiva					
Kantavuusluokka					
Kapillaarisuus					
Maalajin nimi	hkSrMr	hkSrMr	hkSrMr		



Lausunto

16.11.2017

Työnumero	Työn nimi		Pisteen nro
17230	MANTSALA		5
Koordinaatisto	X	Y	Z
ETRS-TM35	6729776.245	412581.193	90.150
Korkeusjärjestelmä	Pohjaveden pinta	Kairauspvm.	Alkukairaus
N2000		14.11.2017	
Kairaustapa		Päättymistapa	
NO - Häiriintynyt näyte		Määräsyvyys	
Kairaaja		Kairauslaite	



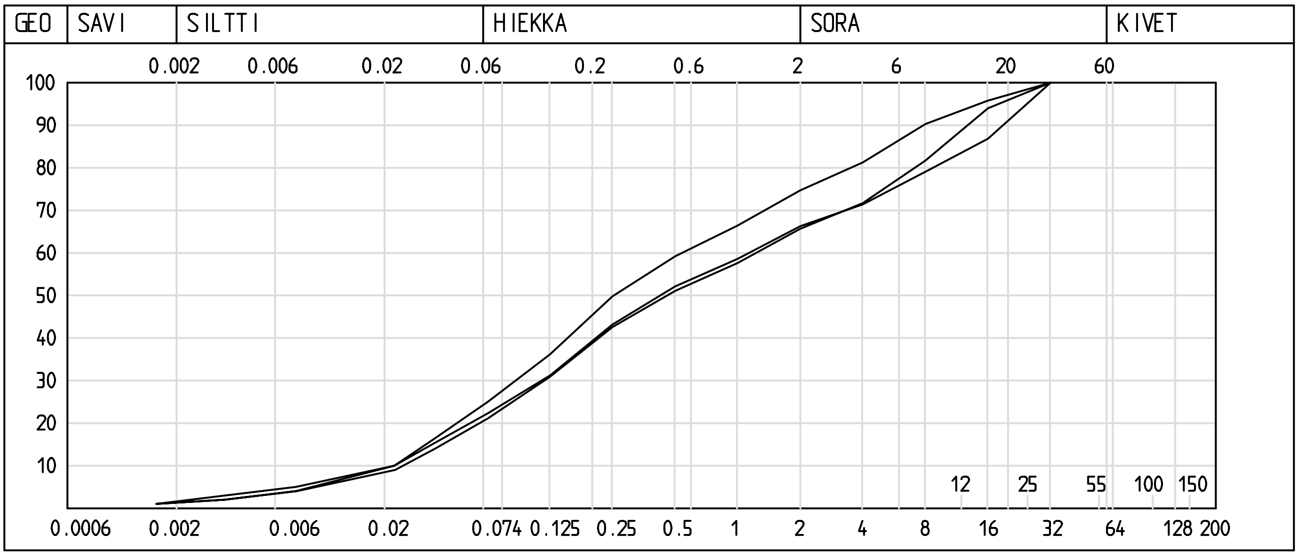
Mittakaava 1:100

LABORATORION TUTKIMUSSELOSTUS

Sivu 1
16.11.2017

Karttalehti	Pisteen nimi	Pisteen nro	Työnumero
	MANTSALA	5	17230
X 6729776.245	Y 412581.193	Z 90.150	
Arkistonumero	Suunnitelmanumero		
Tilaaja		Tutkimus	

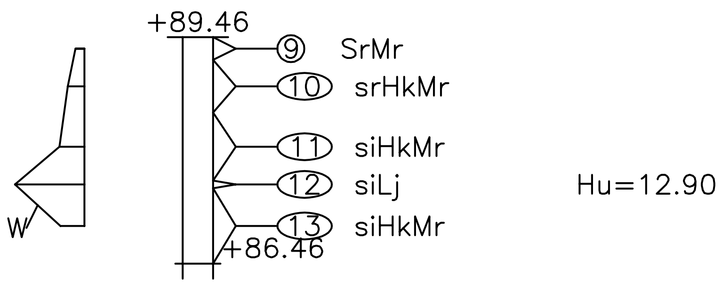
Näytteen tunnus	a	b	c		
Laboratorionumero	6/N0864026	7/N0864027	8/N0864028		
Paalu					
Syvyys	0.25	0.75	1.20		
Korkeustaso	89.90	89.40	88.95		
Ottoaika	14.11.2017	14.11.2017	14.11.2017		
Irtotiheys: kuiva, märkä					
Kiintotiheys					
Vesipitoisuus %	8.2	12.0	10.5		
Humus: poltto, NaOH %					
Routivuus: routimaton, routiva					
Kantavuusluokka					
Kapillaarisuus					
Maalajin nimi	srHkMr	srHkMr	HkMr		



Lausunto

16.11.2017

Työnumero	Työn nimi		Pisteen nro
17230	MANTSALA		7
Koordinaatisto	X	Y	Z
ETRS-TM35	6729793.565	412591.100	89.460
Korkeusjärjestelmä	Pohjaveden pinta	Kairauspvm.	Alkukairaus
N2000		14.11.2017	
Kairaustapa		Päättymistapa	
N□ - Häiriintynyt näyte		Määräsyvyys	
Kairaaja		Kairauslaite	



Mittakaava 1:100

LABORATORION TUTKIMUSSELOSTUS

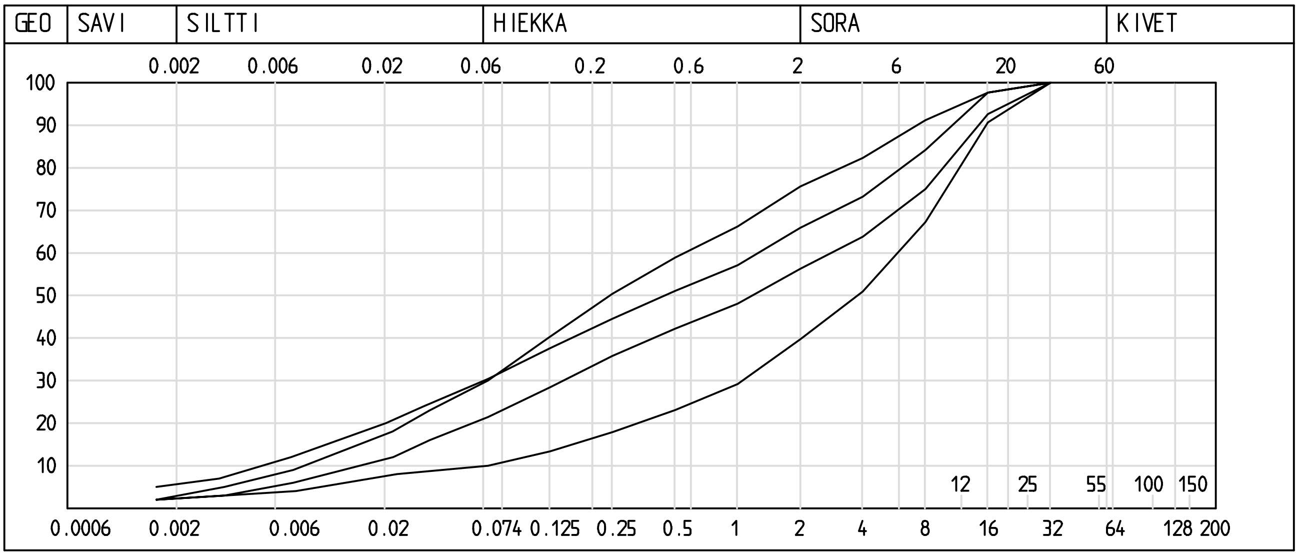
Sivu 1
16.11.2017

Karttalehti	Pisteen nimi	Pisteen nro	Työnumero
	MANTSALA	7	17230

	X	Y	Z	
	6729793.565	412591.100	89.460	
	Arkistonumero	Suunnitelmanumero		

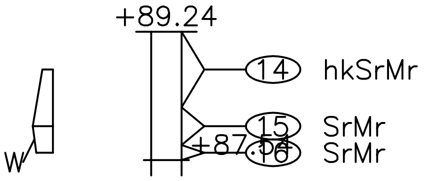
Tilaaja	Tutkimus

Näytteen tunnus	a	b	c	d	e
Laboratorionumero	9/N0864030	10/N0864031	11/N0864032	12/N0864033	13/N0864034
Paalu					
Syvyys	0.15	0.65	1.45	1.95	2.50
Korkeustaso	89.31	88.81	88.01	87.51	86.96
Ottoaika	14.11.2017	14.11.2017	14.11.2017	14.11.2017	14.11.2017
Irtotiheys: kuiva, märkä					
Kiintotiheys					
Vesipitoisuus %	6.0	11.0	16.5	46.0	16.0
Humus: poltto, NaOH %				12.9	
Routivuus: routimaton, routiva					
Kantavuusluokka					
Kapillaarisuus					
Maalajin nimi	SrMr	srHkMr	siHkMr	siLj	siHkMr



Lausunto

Työnumero	Työn nimi		Pisteen nro
17230	MANTSALA		8
Koordinaatisto	X	Y	Z
ETRS-TM35	6729801.079	412595.382	89.240
Korkeusjärjestelmä	Pohjaveden pinta	Kairauspvm.	Alkukairaus
N2000		14.11.2017	
Kairaustapa		Päättymistapa	
N□ - Häiriintynyt näyte		Määräsyvyys	
Kairaaja		Kairauslaite	



Mittakaava 1:100

LABORATORION TUTKIMUSSELOSTUS

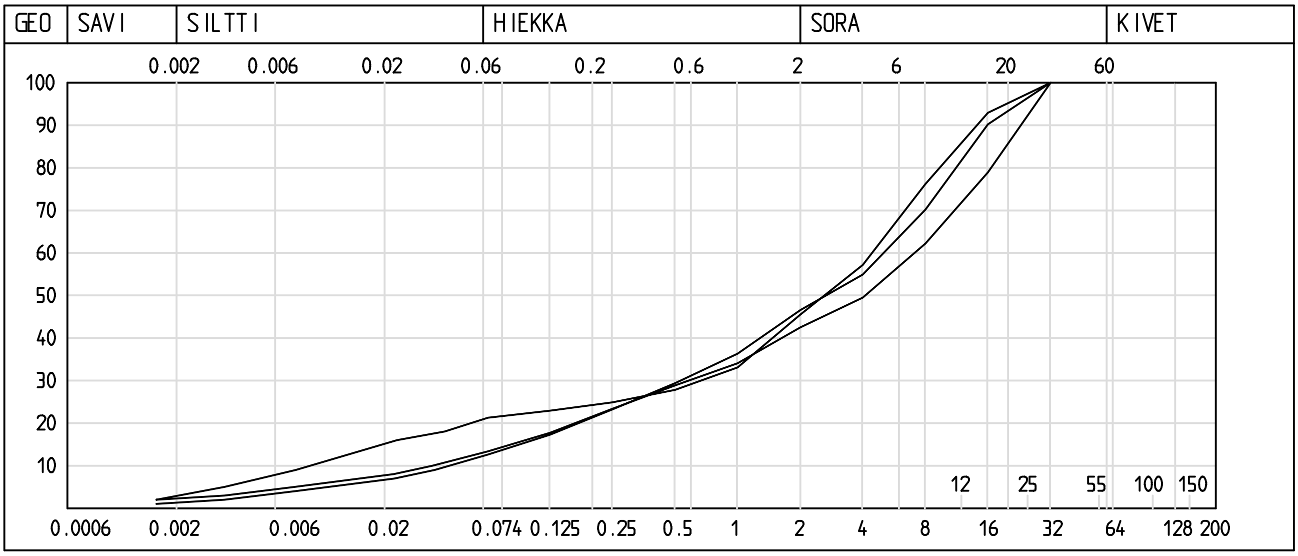
Sivu 1
16.11.2017

Karttalehti	Pisteen nimi	Pisteen nro	Työnumero
	MANTSALA	8	17230

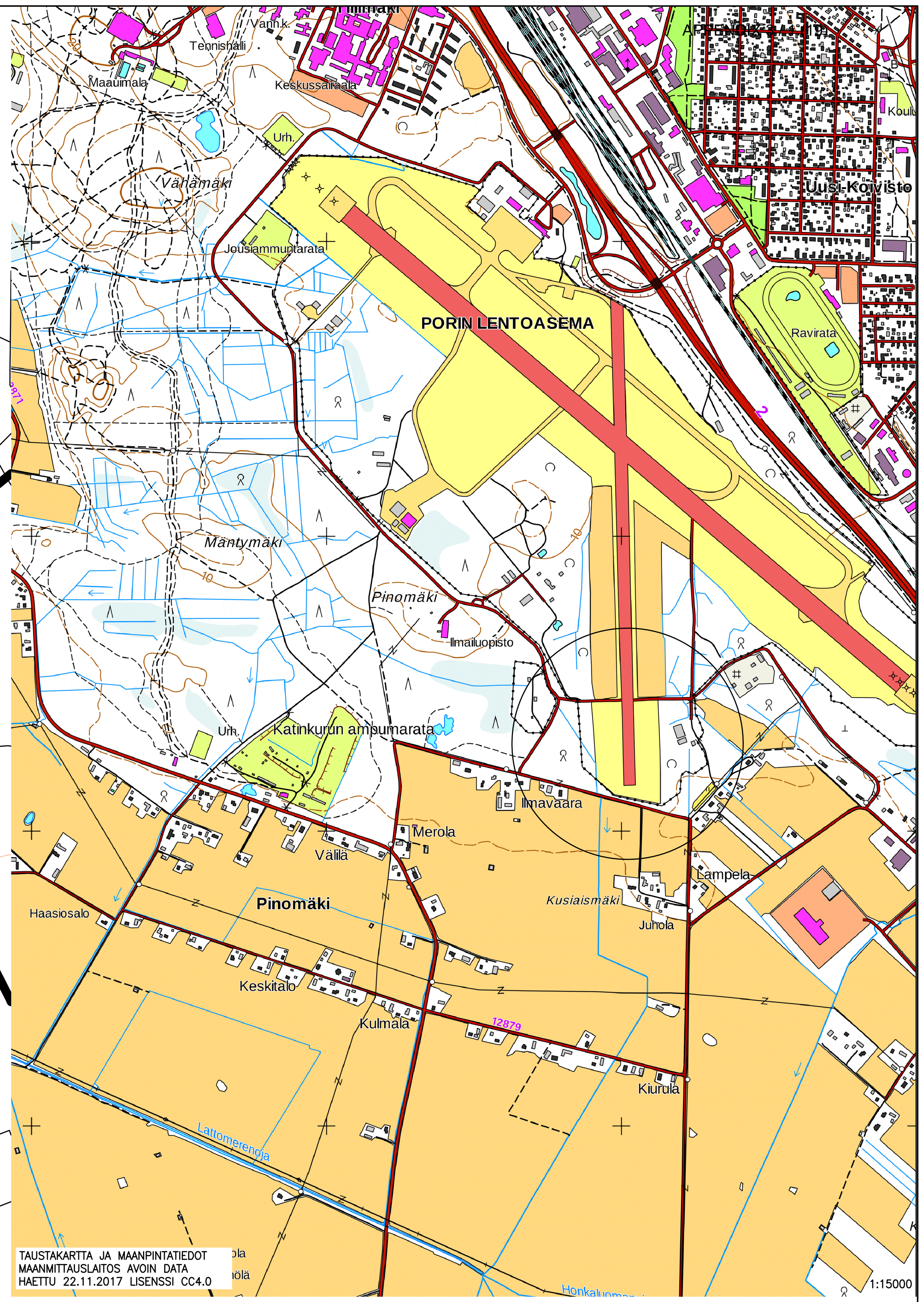
	X	Y	Z	
	6729801.079	412595.382	89.240	
	Arkistonumero	Suunnitelmanumero		

Tilaaja	Tutkimus

Näytteen tunnus	a	b	c		
Laboratorionumero	14/N0864036	15/N0864037	16/N0864038		
Paalu					
Syvyys	0.50	1.25	1.60		
Korkeustaso	88.74	87.99	87.64		
Ottoaika	14.11.2017	14.11.2017	14.11.2017		
Irtotiheys: kuiva, märkä					
Kiintotiheys					
Vesipitoisuus %	7.0	13.5	11.0		
Humus: poltto, NaOH %					
Routivuus: routimaton, routiva					
Kantavuusluokka					
Kapillaarisuus					
Maalajin nimi	hkSrMr	SrMr	SrMr		



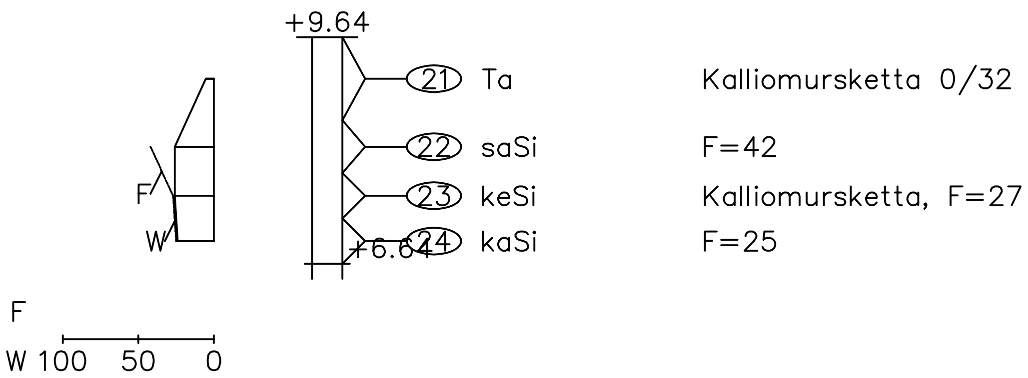
Lausunto



KAUPUNGINOSA / KYLÄ	KORTTELI / TILA	TONTTI / RN:O	ARKISTOINTITUNNUS
RAKENNUSKOHTEEN NIMI: PORIN LENTOASEMA			PIIRUSTUSLAJI
OSOITE			PIIRUSTUKSEN SISÄLTÖ: POHJATUTKIMUSASEMAPIIRROS — MITTAKAAVAT: 1:1000
GEOPALVELU OY MIKKOLANTIE 11 33470 YLÖJÄRVI P (03)2767200 suunnittelu@geopalvelu.fi			
SUUNNITTELIJA	TUTKIJA TV, MA		SUUNNITTELUALA: GEO — TYÖNUMERO: 17230 — PIIRUSTUSNUMERO: 201
HYVÄKSYJÄ	PIIRTÄJÄ ML		
KOORDINAATTI– JA KORKEUSJÄRJESTELMÄ ETRS–TM35FIN			PVM 22.11.2017

21.11.2017

Työnumero	Työn nimi		Pisteen nro
17230	PORI		1
Koordinaatisto	X	Y	Z
ETRS–TM35	6824296.201	223030.181	9.640
Korkeusjärjestelmä	Pohjaveden pinta	Kairauspvm.	Alkukairaus
N2000		21.11.2017	
Kairaustapa		Päättymistapa	
NO – Häiriintynyt näyte		Määräsyvyys	
Kairaaja		Kairauslaite	



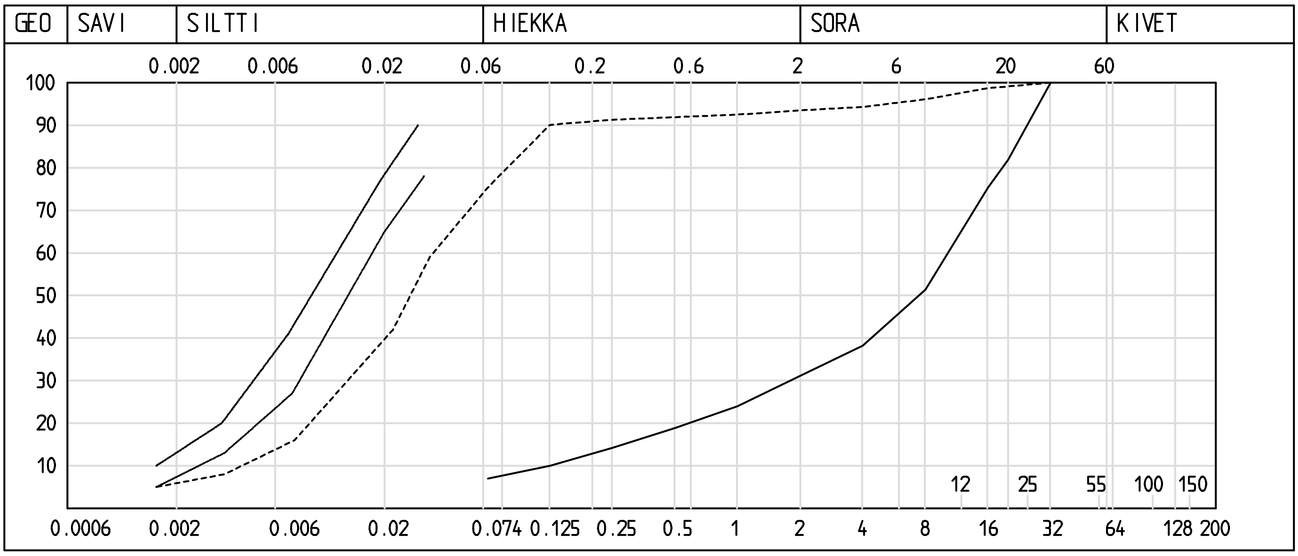
Mittakaava 1:100

LABORATORION TUTKIMUSSELOSTUS

Sivu 1
21.11.2017

Karttalehti	Pisteen nimi	Pisteen nro	Työnumero
	PORI	1	17230
X: 6824296.201	Y: 223030.181	Z: 9.640	
Arkistonumero	Suunnitelmanumero		
Tilaaja		Tutkimus	

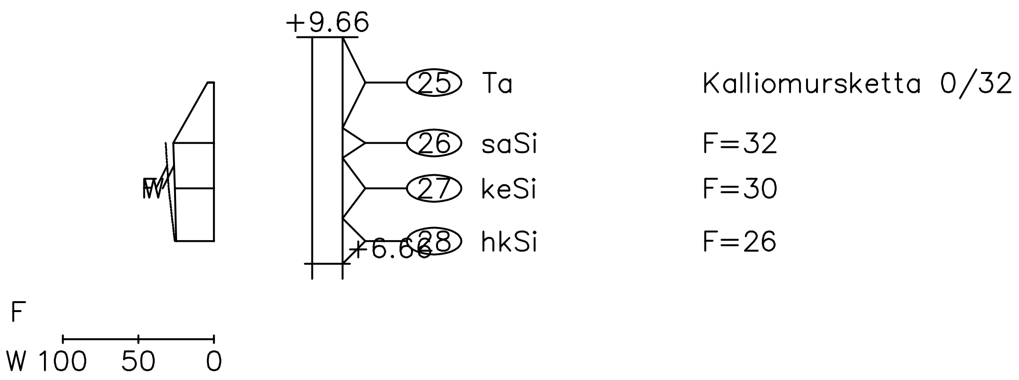
Näytteen tunnus	a	b	c	d	
Laboratorionumero	21/N0865367	22/N0865368	23/N0865369	24/N0865370	
Paalu					
Syvyys	0.55	1.45	2.10	2.70	
Korkeustaso	9.09	8.19	7.54	6.94	
Ottoaika	21.11.2017	21.11.2017	21.11.2017	21.11.2017	
Irtotiheys: kuiva, märkä					
Kiintotiheys					
Vesipitoisuus %	5.3	26.0	26.0	24.0	
Humus: poltto, NaOH %					
Routivuus: routimaton, routiva					
Kantavuusluokka					
Kapillaarisuus					
Maalajin nimi	Ta	saSi	keSi	kaSi	



Lausunto

21.11.2017

Työnumero	Työn nimi		Pisteen nro
17230	PORI		2
Koordinaatisto	X	Y	Z
ETRS–TM35	6824301.360	223028.085	9.660
Korkeusjärjestelmä	Pohjaveden pinta	Kairauspvm.	Alkukairaus
N2000		21.11.2017	
Kairaustapa		Päättymistapa	
NO – Häiriintynyt näyte		Määräsyvyys	
Kairaaja		Kairauslaite	



Mittakaava 1:100

LABORATORION TUTKIMUSSELOSTUS

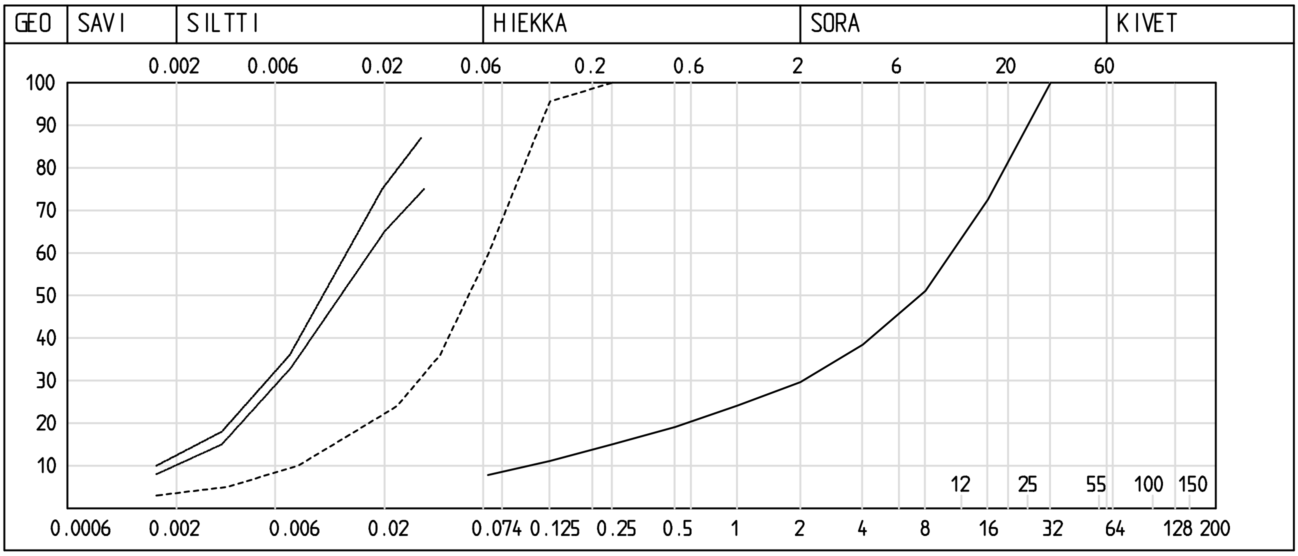
Sivu 1
21.11.2017

Karttalehti	Pisteen nimi	Pisteen nro	Työnumero
	PORI	2	17230

	X	Y	Z	
	6824301.360	223028.085	9.660	
	Arkistonumero	Suunnitelmanumero		

Tilaaja	Tutkimus

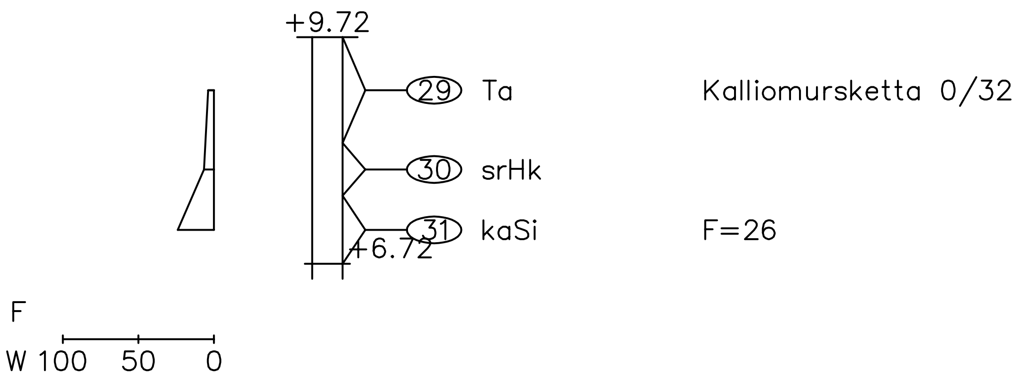
Näytteen tunnus	a	b	c	d	
Laboratorionumero	25/N0865372	26/N0865373	27/N0865374	28/N0865375	
Paalu					
Syvyys	0.60	1.40	2.00	2.70	
Korkeustaso	9.06	8.26	7.66	6.96	
Ottoaika	21.11.2017	21.11.2017	21.11.2017	21.11.2017	
Irtotiheys: kuiva, märkä					
Kiintotiheys					
Vesipitoisuus %	4.3	27.0	26.0	25.0	
Humus: poltto, NaOH %					
Routivuus: routimaton, routiva					
Kantavuusluokka					
Kapillaarisuus					
Maalajin nimi	Ta	saSi	keSi	hkSi	



Lausunto

21.11.2017

Työnumero 17230	Työn nimi PORI		Pisteen nro 4
Koordinaatisto ETRS–TM35	X 6824312.525	Y 223023.623	Z 9.720
Korkeusjärjestelmä N2000	Pohjaveden pinta	Kairauspvm. 21.11.2017	Alkukairaus
Kairaustapa NO – Häiriintynyt näyte		Päättymistapa Määräsyvyys	
Kairaaja		Kairauslaite	



Mittakaava 1:100

LABORATORION TUTKIMUSSELOSTUS

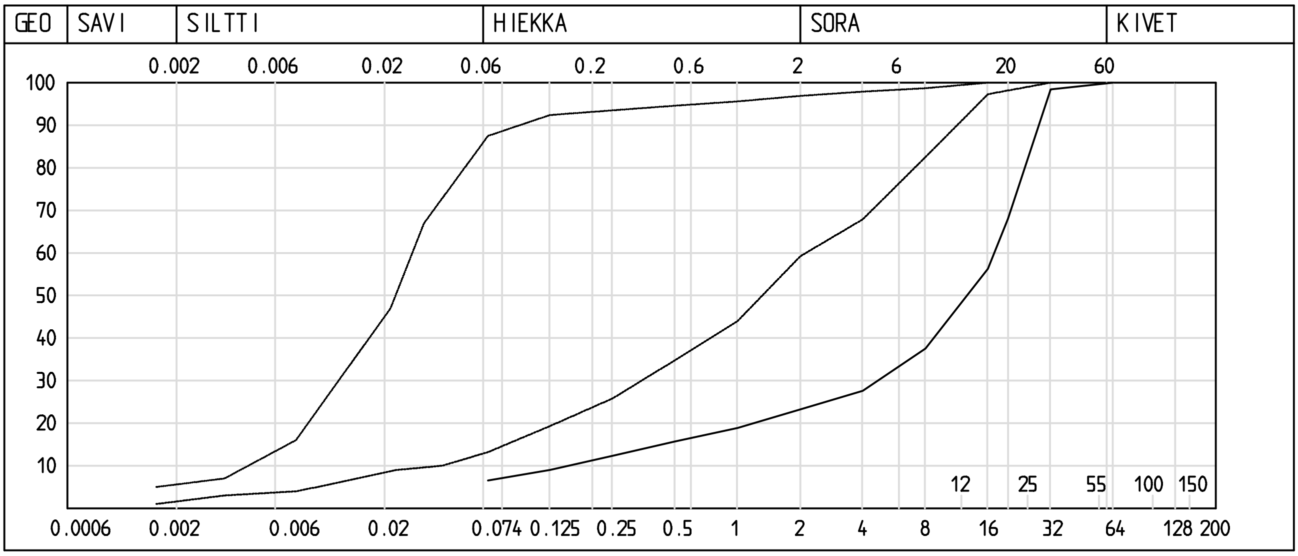
Sivu 1
21.11.2017

Karttalehti	Pisteen nimi	Pisteen nro	Työnumero
	PORI	4	17230

	X	Y	Z	
	6824312.525	223023.623	9.720	
	Arkistonumero	Suunnitelmanumero		

Tilaaja	Tutkimus

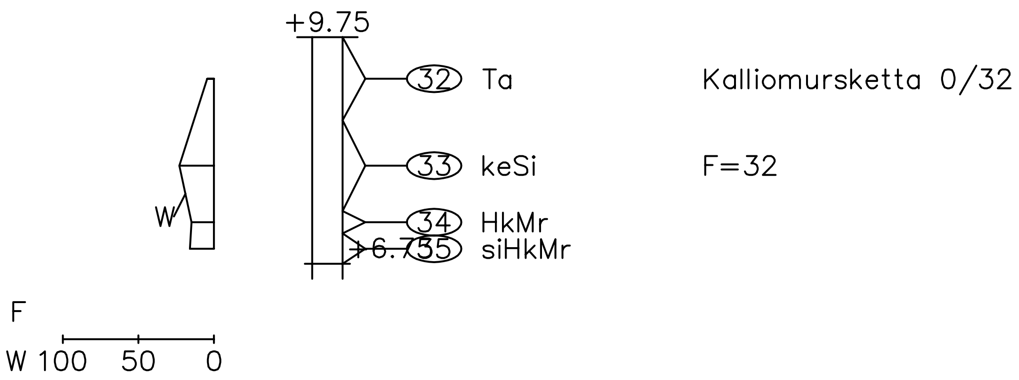
Näytteen tunnus	a	b	c		
Laboratorionumero	29/N0865377	30/N0865378	31/N0865379		
Paalu					
Syvyys	0.70	1.75	2.55		
Korkeustaso	9.02	7.97	7.17		
Ottoaika	21.11.2017	21.11.2017	21.11.2017		
Irtotiheys: kuiva, märkä					
Kiintotiheys					
Vesipitoisuus %	3.8	6.7	24.0		
Humus: poltto, NaOH %					
Routivuus: routimaton, routiva					
Kantavuusluokka					
Kapillaarisuus					
Maalajin nimi	Ta	srHk	kaSi		



Lausunto

21.11.2017

Työnumero	Työn nimi		Pisteen nro
17230	PORI		6
Koordinaatisto	X	Y	Z
ETRS–TM35	6824321.974	223019.763	9.750
Korkeusjärjestelmä	Pohjaveden pinta	Kairauspvm.	Alkukairaus
N2000		21.11.2017	
Kairaustapa		Päättymistapa	
NO – Häiriintynyt näyte		Määräsyvyys	
Kairaaja		Kairauslaite	



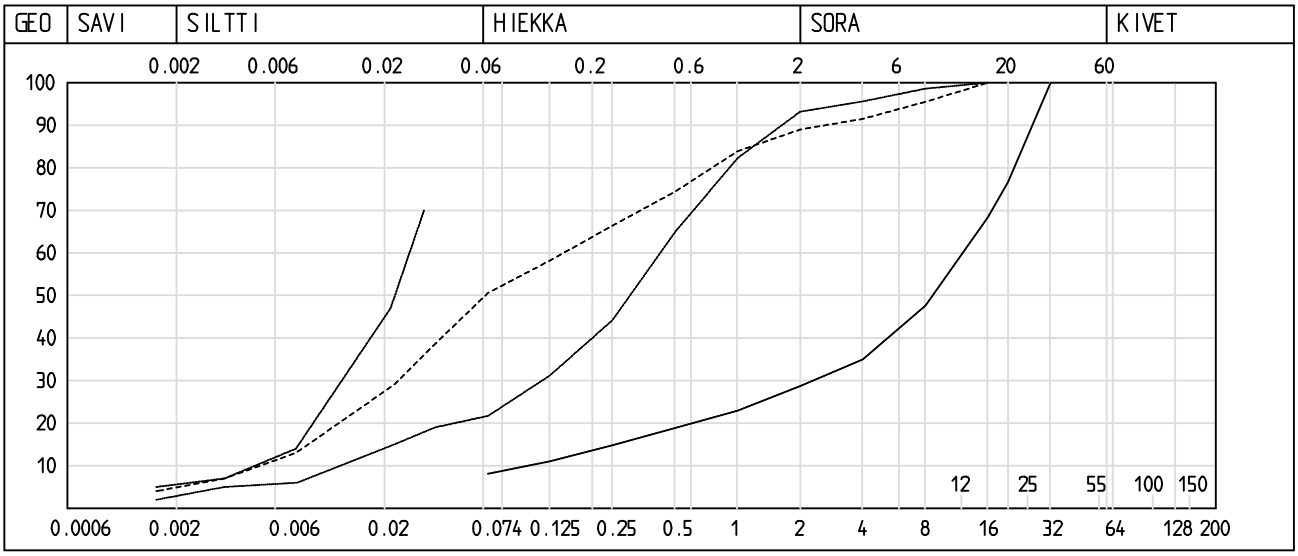
Mittakaava 1:100

LABORATORION TUTKIMUSSELOSTUS

Sivu 1
21.11.2017

Karttalehti		Pisteen nimi	Pisteen nro	Työnumero
		PORI	6	17230
	X 6824321.974	Y 223019.763	Z 9.750	
	Arkistonumero	Suunnitelmanumero		
Tilaaja			Tutkimus	

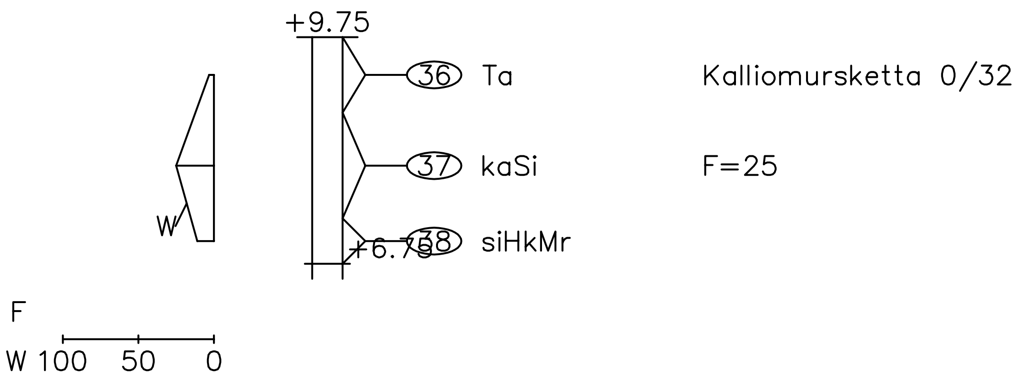
Näytteen tunnus	a	b	c	d	
Laboratorionumero	32/N0865381	33/N0865382	34/N0865383	35/N0865384	
Paalu					
Syvyys	0.55	1.70	2.45	2.80	
Korkeustaso	9.20	8.05	7.30	6.95	
Ottoaika	21.11.2017	21.11.2017	21.11.2017	21.11.2017	
Irtotiheys: kuiva, märkä					
Kiintotiheys					
Vesipitoisuus %	4.5	23.0	15.0	16.0	
Humus: poltto, NaOH %					
Routivuus: routimaton, routiva					
Kantavuusluokka					
Kapillaarisuus					
Maalajin nimi	Ta	keSi	HkMr	siHkMr	



Lausunto

21.11.2017

Työnumero	Työn nimi		Pisteen nro
17230	PORI		7
Koordinaatisto	X	Y	Z
ETRS–TM35	6824326.340	223017.865	9.750
Korkeusjärjestelmä	Pohjaveden pinta	Kairauspvm.	Alkukairaus
N2000		21.11.2017	
Kairaustapa		Päättymistapa	
NO – Häiriintynyt näyte		Määräsyvyys	
Kairaaja		Kairauslaite	



Mittakaava 1:100

LABORATORION TUTKIMUSSELOSTUS

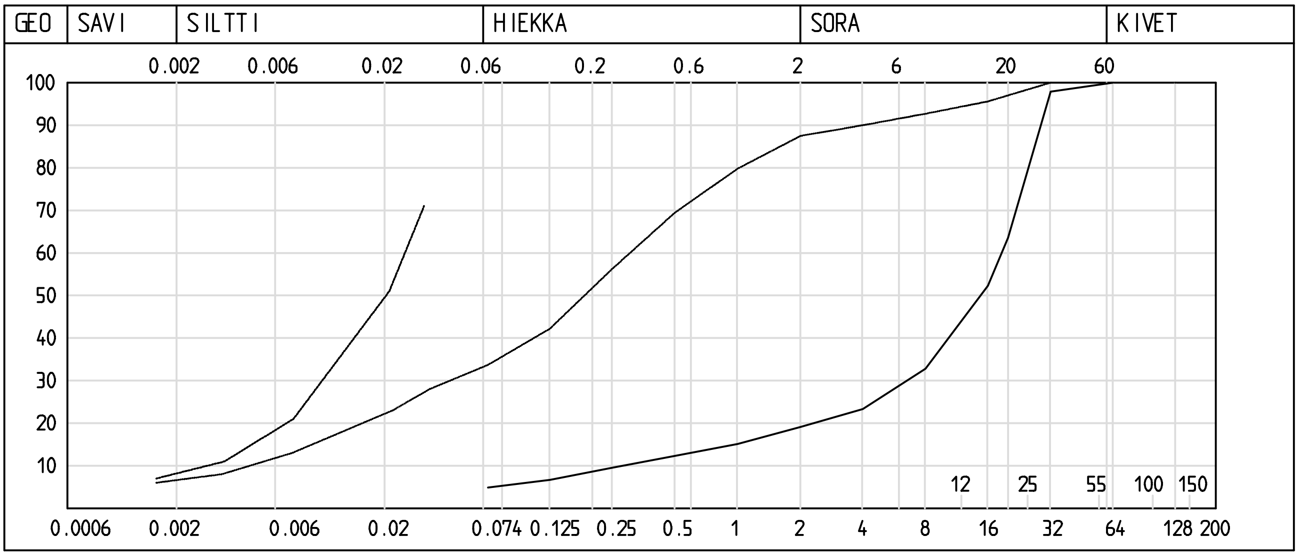
Sivu 1
21.11.2017

Karttalehti	Pisteen nimi	Pisteen nro	Työnumero
	PORI	7	17230

	X	Y	Z	
	6824326.340	223017.865	9.750	
	Arkistonumero	Suunnitelmanumero		

Tilaaja	Tutkimus

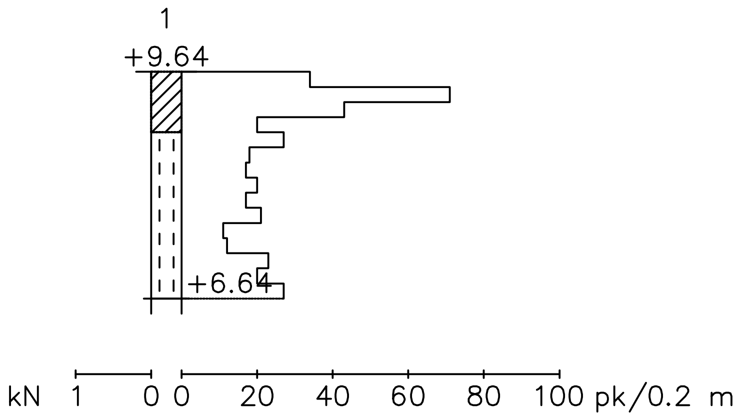
Näytteen tunnus	a	b	c		
Laboratorionumero	36/N0865386	37/N0865387	38/N0865388		
Paalu					
Syvyys	0.50	1.70	2.70		
Korkeustaso	9.25	8.05	7.05		
Ottoaika	21.11.2017	21.11.2017	21.11.2017		
Irtotiheys: kuiva, märkä					
Kiintotiheys					
Vesipitoisuus %	3.2	25.0	11.0		
Humus: poltto, NaOH %					
Routivuus: routimaton, routiva					
Kantavuusluokka					
Kapillaarisuus					
Maalajin nimi	Ta	kaSi	siHkMr		



Lausunto

20.11.2017

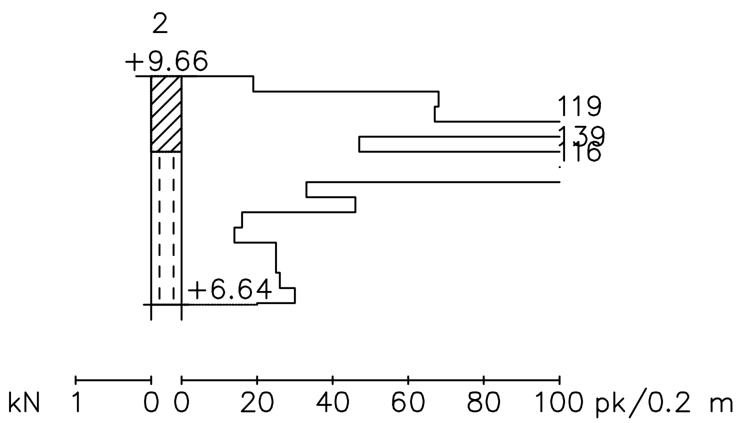
Työnumero	Työn nimi		Pisteen nro
17230	PORI		1
Koordinaatisto	X	Y	Z
ETRS–TM35	6824296.201	223030.181	9.640
Korkeusjärjestelmä	Pohjaveden pinta	Kairauspvm.	Alkukairaus
N2000		13.11.2017	–
Kairaustapa		Päättymistapa	
PA – Painokairaus		Määräsyvyys	
Kairaaja		Kairauslaite	



Mittakaava 1:100

20.11.2017

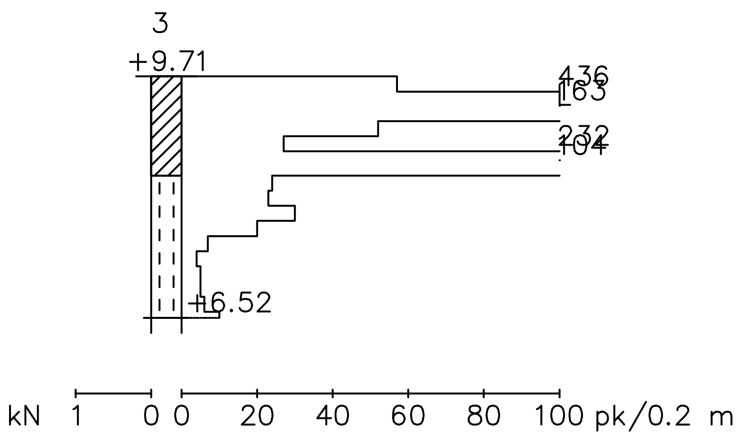
Työnumero	Työn nimi		Pisteen nro
17230	PORI		2
Koordinaatisto	X	Y	Z
ETRS-TM35	6824301.360	223028.085	9.660
Korkeusjärjestelmä	Pohjaveden pinta	Kairauspvm.	Alkukairaus
N2000		13.11.2017	–
Kairaustapa		Päättymistapa	
PA – Painokairaus		Määräsyvyys	
Kairaaja		Kairauslaite	



Mittakaava 1:100

20.11.2017

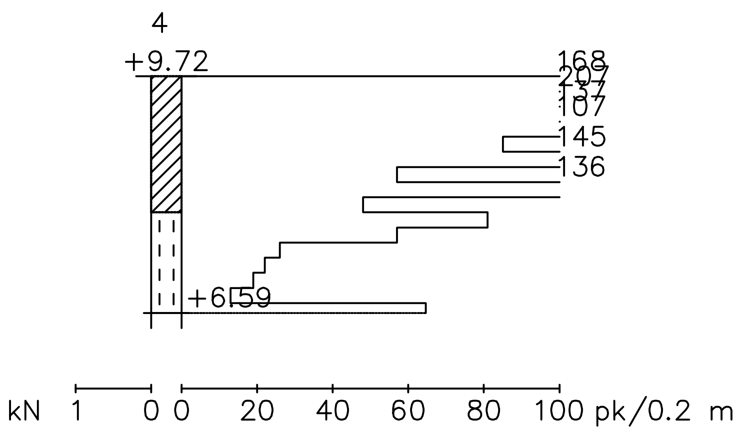
Työnumero	Työn nimi		Pisteen nro
17230	PORI		3
Koordinaatisto	X	Y	Z
ETRS–TM35	6824306.885	223025.793	9.710
Korkeusjärjestelmä	Pohjaveden pinta	Kairauspvm.	Alkukairaus
N2000		13.11.2017	–
Kairaustapa		Päättymistapa	
PA – Painokairaus		Määräsyvyys	
Kairaaja		Kairauslaite	



Mittakaava 1:100

20.11.2017

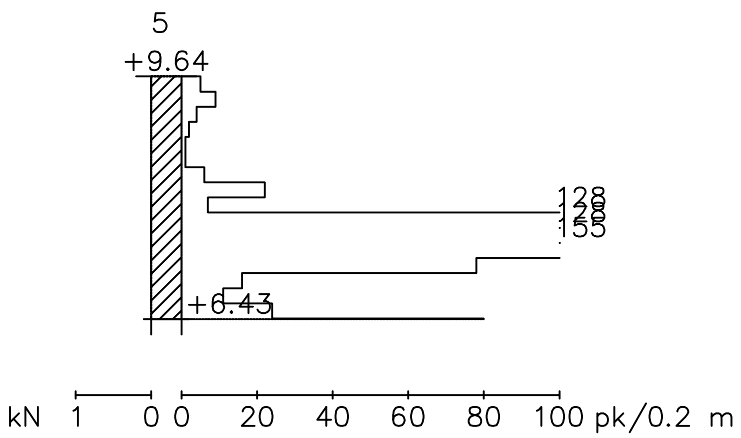
Työnumero	Työn nimi		Pisteen nro
17230	PORI		4
Koordinaatisto ETRS–TM35	X 6824312.525	Y 223023.623	Z 9.720
Korkeusjärjestelmä N2000	Pohjaveden pinta	Kairauspvm. 13.11.2017	Alkukairaus –
Kairaustapa PA – Painokairaus		Päättymistapa Määräsyvyys	
Kairaaja		Kairauslaite	



Mittakaava 1:100

20.11.2017

Työnumero	Työn nimi		Pisteen nro
17230	PORI		5
Koordinaatisto	X	Y	Z
ETRS–TM35	6824319.291	223022.458	9.640
Korkeusjärjestelmä	Pohjaveden pinta	Kairauspvm.	Alkukairaus
N2000		13.11.2017	–
Kairaustapa		Päättymistapa	
PA – Painokairaus		Määräsyvyys	
Kairaaja		Kairauslaite	



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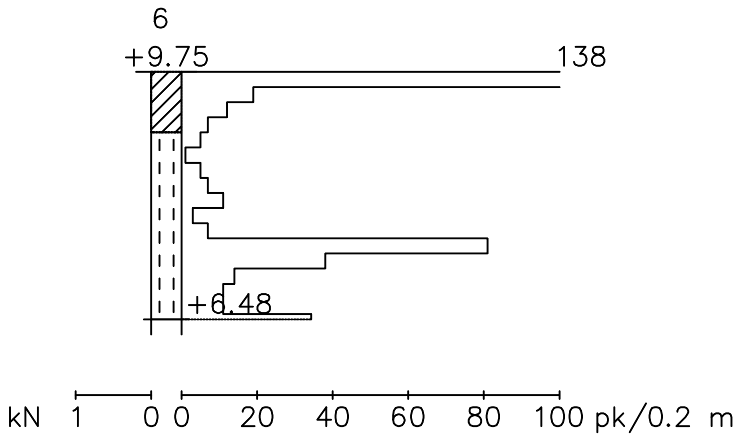
20.11.2017

Työnumero	Työn nimi		Pisteen nro
17230	PORI		6
Koordinaatisto	X	Y	Z
ETRS–TM35	6824321.974	223019.763	9.750
Korkeusjärjestelmä	Pohjaveden pinta	Kairauspvm.	Alkukairaus
N2000		13.11.2017	–
Kairaustapa		Päättymistapa	
PA – Painokairaus		Määräsyvyys	
Kairaaja		Kairauslaite	



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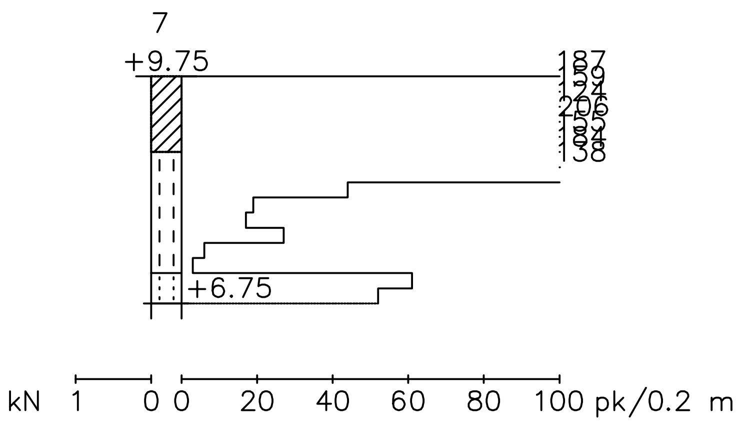
20.11.2017

Työnumero	Työn nimi		Pisteen nro
17230	PORI		7
Koordinaatisto	X	Y	Z
ETRS–TM35	6824326.340	223017.865	9.750
Korkeusjärjestelmä	Pohjaveden pinta	Kairauspvm.	Alkukairaus
N2000		13.11.2017	–
Kairaustapa		Päättymistapa	
PA – Painokairaus		Määräsyvyys	
Kairaaja		Kairauslaite	



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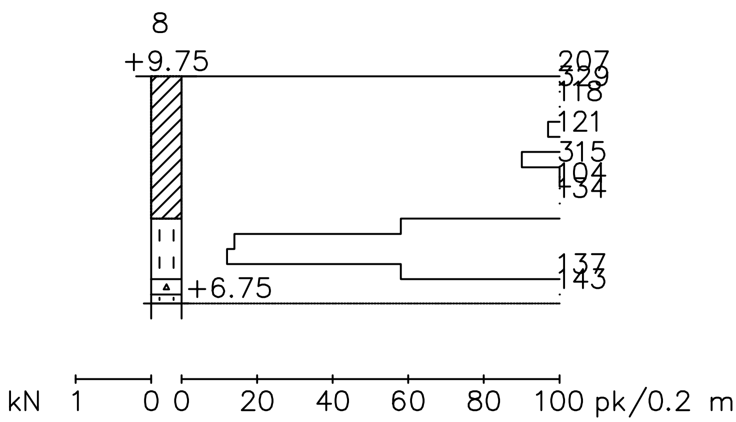
20.11.2017

Työnumero 17230	Työn nimi PORI		Pisteen nro 8
Koordinaatisto ETRS–TM35	X 6824330.100	Y 223016.225	Z 9.750
Korkeusjärjestelmä N2000	Pohjaveden pinta	Kairauspvm. 13.11.2017	Alkukairaus –
Kairaustapa PA – Painokairaus		Päättymistapa Määräsyvyys	
Kairaaja		Kairauslaite	



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